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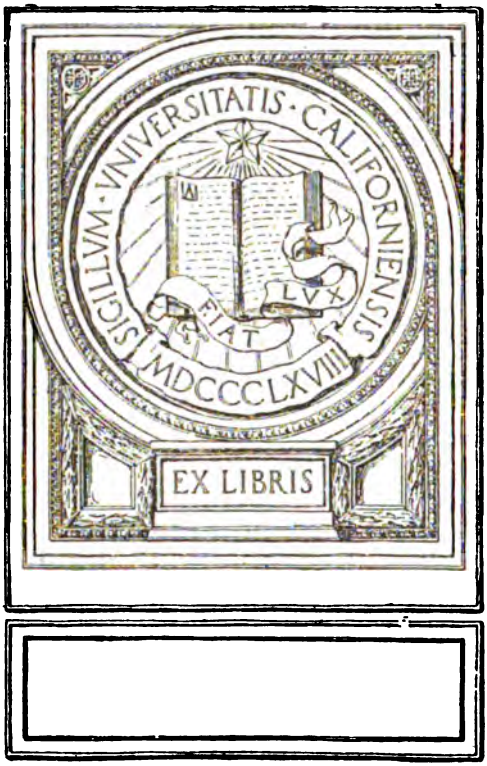

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THE PRACTICE OF RAILWAY SURVEYING AND PERMANENT WAY WORK

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THE PRACTICE OF RAILWAY SURVEYING AND PERMANENT WAY WORK

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ILLUSTRATED

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PREFACE

THE object aimed at in this volume is to deal mainly with the practical side of Railway Surveying and Permanent Way Construction, a knowledge of Surveying in general being assumed.

The authors gained their experience as members of the staffs of two leading English Railways, and also when carrying out railway work abroad. The knowledge thus acquired of the best English practice, as well as of the methods adopted in the construction of economic railways in unexplored and undeveloped countries, has enabled them to deal with the subject from the practical point of view. Many of the methods described are original; some are traditional among Railway Engineers; and some are those adopted by American Engineers, acquaintance with which is necessary for the more complete study of the subject. Little or none of the matter included under "Permanent Way" has previously appeared in print, and it is hoped that this will be of much assistance to the young permanent way engineer.

The authors are indebted to Mr. W. H. Shortt for matter from his paper "Practical Method for the Improvement of Existing Railway Curves," read at the Institution of Civil Engineers on January 12th, 1909, and to Mr. James Glover for the mathematical treatment of Transition Curves from his paper "Transition Curves for Railways" in the Minutes of Proceedings of the Inst. C.E., vol. cxl. p. 161.

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THE PRACTICE OF RAILWAY SURVEYING AND PERMANENT WAY WORK

CHAPTER I

CLASSIFICATION OF RAILWAYS

FROM the highest class of main trunk line with its gentle gradients, long lengths of straight, and its flat curves, over which fast and heavy traffic is hauled with ease, to the cheap surface line or light railway with its steep gradients and frequent and sharp curves, railways are to be found graduating through every intermediate stage without any break to form a line of demarcation. But for convenience and, as it were, to fix some "landmarks" from which to study further refinements of classification they may be divided into three classes.

(1) **The Main or Trunk Line.**—This, somewhat like the main river of a large drainage basin into which all tributaries flow, is the great collector and distributor of traffic, and as such must be able to cope with the ever-increasing flow of traffic from its tributary lines. Thus it happens that the whole railway system is dependent upon the main line for its operating efficiency. The greater the traffic over any part of the road the greater becomes the need for a high-class working line. Lines through thickly-populated districts, through large industrial areas, whether manufacturing or mining, or connecting such districts with commercial seaports, require to be of a high class in order to cope adequately with the heavy freight and passenger traffic. But such lines anticipate already settled country.

The development or partial development of new countries rich in minerals or agricultural lands will require the main line of railway to be of a quality dependent upon the rate of development anticipated. Trunk lines in the United States and Canada, which were laid through country with little or no habitation, had to depend for their traffic upon the development resulting from the existence of the railways themselves. Such development and consequently such traffic cannot be very closely estimated for any particular time. A country which has to depend upon the railway, in the first instance, for its inhabitants, may be very

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slow in developing and in providing sufficient traffic to make the railway a paying concern. On this account some of the great pioneer roads, though they were main lines and intended as such, had to be built with the anticipation of little traffic for many years. The Canadian Pacific Railway is probably the best example of such a line. In this case, in order to connect the rich agricultural and mineral areas together and with the seaports, enormous lengths of line had to be built. The cost of this work was colossal, yet not nearly as great as it would have been if a high-class main line had been constructed. It is a principle of railway construction in America to expend no more on the construction of a line than the traffic within a few years is likely to justify, and when the volume of traffic requires a better line to make a further outlay in carrying out the needed improvements.

The surveying of a cheap main line with a view to its subsequent improvement is work which can only be performed by a capable railway engineer. The line has to be built cheaply, but it will ultimately need to be high class. Economies may be practised by substituting for long embankments wood trestle viaducts, and heavy cuttings may be materially lightened by making steeper gradients. At a subsequent time when the traffic justifies it this steep gradient may be improved by additional excavation and the material possibly help to fill an embankment on a site previously occupied by a trestle viaduct. This is only one example of what can be done, but if subsequent improvements are kept in view when the first line is constructed, they can be performed more economically when the time for them arrives. It is difficult to build a main line and expect it to pay for itself from the beginning. Even the inferior gradients of a cheap main line are superior in most cases to those which must be adopted if the line is to pay its way from the first.

In the case of main lines the matter of investigating the amount of traffic likely to be obtained in certain districts is not so much dealt with by the locating engineer, since the main points to be connected have been decided upon usually already. But in the case of making a choice between two routes of somewhat equal merit a decision may be arrived at on the traffic basis.

It is not always economical to improve the existing line. Sometimes it is found cheaper to construct a new line in a different location which provides the better-class curves and gradients required, and at the same time opens up new country. The original Canadian Pacific main line from Montreal to Toronto, *viâ* Peterborough, is through country so difficult that the cost of improving the line and obtaining a much better ruling gradient, which was very necessary, to cope with the rapidly growing traffic, proved too heavy and an entirely new line, having first-class gradients, was built through other country to take the place of the

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difficult section of the original line, in spite of its having to compete for much of its length with two existing roads.

Although a main line is expected at many points to have branch lines feeding it, it is false economy to construct it always through the easiest country, leaving to branch lines the tapping of the most fertile or rich mineral areas. If it is possible without seriously spoiling the gradients or alignment to bring the main line through these areas, the saving in the working of branch lines may be effected.

(2) **Branch Lines and Second-class Railways.**—In the British Isles second-class lines will pass through country only moderately populated and with fewer industries. Their goods and passenger traffic will be derived from small towns or villages frequently at considerable distances apart. Such lines are naturally very important for the districts through which they pass and often bring about the development of some into large and important business centres. But prior to that their traffic is not sufficient to qualify the road for those improvements which raise main lines to a high standard.

These second-class lines in a country like Canada can only be constructed very cheaply, and then may have to exist for years on small profits until the district through which it passes becomes more developed. But branch lines are important feeders to the main line and can often be operated at little additional cost to the system. Even though a branch line may not in itself appear to pay, the benefit to the whole system resulting from it is sufficient to justify its existence.

As the object of the branch line is to bring business to the main line from some town or industrial area, there is no object in making it longer than the length necessary to connect directly the town with the main line. Consequently, if the topography permits, the line should be laid in a direction as nearly square to the main line as possible, only turning off to connect to the latter by a curve. An important exception to this rule is when a town on the main line is connected to a town at some distance from it. In that case the branch line should take the most direct route between the two towns and not first strike the main line by the shortest route. A direct line encourages traffic between two towns where an indirect one often fails to do so to any important extent. Again, sidings will exist on the main line at the town and can be used by the branch line also, and the junction can be worked there with no additional expense.

The gradients adopted on branch lines are always much steeper than on the main line, as the freight is hauled in smaller train loads and for shorter distances. It is a great mistake to compare the cost of hauling on branch lines with that on the main line. The main line is the great machine for economic hauling upon which railway systems depend for making their profits, but without the branch lines to supply the main

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line little freight would be provided, upon the hauling of which the main line could prove its efficiency. On this account industrial areas anywhere within reasonable distance from the main line must be connected with it, if the community in that area is at all progressive. Even through very difficult country the branch line ought to be constructed, and consequently steep gradients are not infrequent on such lines.

In the case of a new railway system, the selection of points at which branch lines should be constructed should be entrusted to one familiar with the districts through which it passes and capable of arriving at the relative values of the business likely to be provided for the railway by each community under consideration. Locating engineers are sometimes required to report upon the probable volume of traffic to be derived from branch lines through certain districts, but their estimates must be frequently wide of the mark. Freight agents or others whose business is to deal constantly with the procuring of traffic should be able to perform this duty much more satisfactorily.

(3) **Third-class Roads and Light Railways.**—The branch lines referred to above as having very steep gradients really belong to this class. They are constructed through poorer districts and such as provide less traffic or run a considerable distance to a factory or a small mining centre. They are more or less surface roads, and sometimes are constructed with a narrower gauge than the main line.

In the United Kingdom light railways would come under this category. Those which are laid out partly on the high roads and partly almost on the surface through fields cost very little to construct, but their gradients and curves prohibit a high speed. Quarries, mines and factories sometimes build narrower gauge (from about 2 feet upwards) lines for their own convenience, and also to connect them with the nearest railway. Such lines suffer from the inconvenience that their rolling stock cannot be taken on to the main line, necessitating the transhipment of all freight unless the trucks are so constructed that they can be lifted off their wheel frames by a crane and placed thus loaded on the main line cars. The limiting gradients used on some of these lines are extremely steep and the curves nearly as sharp as those adopted by street-car lines.

General Remarks.—The need for so many classes or graduations of classes of railways can be found in the fact that the expenditure on each railway should be in proportion to the traffic expected. Consequently, even in easy country, those refinements in grading which may be carried out with little additional cost and convert the road into a high-class one, are often omitted because the probable traffic is not sufficient to warrant even this small increase in expenditure. Again, in difficult country if

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the traffic prospects are no better the expenditure should not be any heavier, but the resulting road will be inferior. Thus here the same condition of traffic provides two different types of roads because the topographical conditions of the country in each case is different. Now it is only necessary to vary the traffic prospects in each case as well and further classes of roads will result.

When starting a survey for a railway the locating engineer must know approximately the average expenditure per mile at his disposal, and from that be able to determine on the site the best class of road he may expect to build. Any improvement in the line at first located which reduces the annual operating expenses by an amount greater than the annual interest on the capital so expended on improvement must be carefully considered and carried out if possible. It is in paying attention to these problems throughout the whole survey that the locating engineer arrives at the most suitable and economic line. A simple instance will exemplify this. A new line is being surveyed running east and west. The heavy traffic has to be hauled east and very light traffic west. Two sections of the line have ruling gradients of ·4 per cent. (1 in 250) and ·6 per cent. (1 in 166) respectively against the east-bound traffic according to the first location. There are 3 miles of ·6 per cent. gradient in the latter section, which is 25 miles long. Using the units adopted in America to simplify the calculations the following may be assumed :—

That with modern rolling stock and heavy engines 2000 tons can be hauled up a ·4 per cent. gradient and 1400 tons can be hauled up a ·6 per cent. gradient.

That for purposes of calculation the cost per train-mile may be taken as $1·0.

If the 3 miles of ·6 per cent. gradient can be reduced to ·4 per cent. at a cost of 5 miles more of distance at $50,000 per mile, the annual cost of operating each line will throw useful light on such improvements and their cost.

Assuming that 4,000,000 tons per annum is hauled eastwards. The cost of operating each line is as follows :—

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Operating the section of line when ·6 per cent. is the ruling gradient—

$\frac{4000000}{1400} \times 25 \times \$1 = \$74,286$ = cost of east-bound traffic.
The same trains hauled westward = \$74,286 = cost of west-bound traffic.

Annual operating expenses = \$148,572

Operating the section of line when ·4 per cent. is its ruling gradient—

$\frac{4000000}{2000} \times 30 \times \$1 = \$60,000$
is cost of east-bound traffic.
The same trains hauled westward = \$60,000
is cost of west-bound traffic.

——— 120,000

∴ Difference in cost of operating = \$28,000

But the cost of additional 5 miles of line at \$50,000 per mile is \$250,000 and the interest on this at 4 per cent. = 10,000

Annual saving from the improved gradient = \$18,000

It is assumed above that the west-bound traffic, which is light, can be hauled against any gradients existing in that direction.

The saving of \$18,000 annually by adopting this improvement suggests that even if the additional 5 miles of line was constructed at greater cost per mile than \$50,000 the margin in favour of the improvement is still considerable. Again, assuming that money can be borrowed at the rate of 4 per cent. annual interest, \$18,000 will pay the interest on \$450,000 which latter sum can be expended, in addition, on the construction of the 5 miles or \$90,000 per mile, making in all the cost of constructing the additional 5 miles, \$140,000 per mile, if necessary. In this way the limits within which it is economical to alter a ruling gradient in favour of a better one are shown clearly, provided the assumptions of the amount of traffic and the cost per train-mile can be relied upon and the ratios of tons haulable on the respective ruling gradients.

Many problems of choice between routes or the improvement of routes can be solved in this way by estimating the cost of operation under both conditions, but some experience is necessary in estimating the amount of present, probable or ultimate traffic to be derived from the district. But the great problem which confronts the locating engineer is that of selecting the most suitable strip of country within which to attempt to locate a satisfactory line. The study of the terrain, with its ever-changing surface, for this purpose, is carried out during the reconnaissance surveys, particulars of which are given in the next chapter.

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CHAPTER II

RAILWAY EXPLORATION OR RECONNAISSANCE

Preliminary Reconnaissance

Preliminary explorations are first carried out by an experienced railway engineer, and the results are then made the basis of an estimate for submission to the directors or others representing the financial interests of the proposed line.

On such a reconnaissance observations are taken of the agricultural possibilities of the country and of any evident prospects of mineral development, the population is noted, the amount and nature of the industries, and the topographical features; and comparing these with districts which have already been developed by railways, an estimate of the probable volume of traffic both present and future, as well as the type of line and its various characteristics, and the approximate cost of construction and operation is made.

But reconnaissance for route which is usually subsequent to the above requires to be put into practice as soon as it has been decided to construct a railway, if feasible, through the district.

The extent of the work needed for such surveys varies enormously with the nature of the country and the quality of its available maps. Thus in a densely-populated country the object of the railway would be lost unless it passed through certain towns or "governing points." And if these should be fairly close together the deviations of the line are enclosed between narrow limits. Further, as excellent topographical maps of such districts frequently exist, it is possible to place the line at once within still narrower limits. Such conditions usually exist in this country with our dense population and our ordnance plans giving 100-foot contour lines. In such cases the reconnaissance survey means little more than sketching the approximate position of the railway on the ordnance plan in the office before starting over the ground, and then finding the position of, and walking along the course of, this line on the ground, for the purpose of further sketching on the plan any improvements which the site reveals. This is necessary as 100-foot contour lines

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are not sufficiently close to each other to eliminate entirely the need of reconnaissance. (The maps of the United States Geological Survey are usually to a scale of 1 inch to the mile with contours at 20-foot intervals, but are not yet prepared for the whole of the country.) This closer approximation to the best position of the line to be subsequently surveyed is of inestimable value, because any effort to deviate sufficiently from the approximate route in the subsequent or location surveys, can only be done at considerable expense and loss of time; and the actual cost of such reconnaissance surveys is very small.

Again, in more sparsely-populated districts, although good maps may be obtainable, to simplify this part of the work materially the governing points are probably much farther apart, so giving a wider area within which to find the most economical location of the line, and consequently involving a more extensive examination of the country during reconnaissance between these governing points. But it must not be supposed that towns or factories or other industrial enterprises are the only governing points to be considered in railway exploration surveys. Although the earning capacity of a railway is a very important factor, the cost of construction is also of primary importance. The topography of the country must be studied, and such points as the lowest pass through a range of mountains or the narrowest point of a deep river gorge are essential governing points.

In conducting a reconnaissance along the probable line through such a district, although good maps will enable a fair approximation to the best position for the road to be fixed in the office, the actual work in revising that line when on the ground will be much more arduous than in the previous case, as a wider belt of country will require to be examined.

Again, if the maps of this sparsely-inhabited country be very poor, giving very little natural or artificial topography, it may be impossible to sketch a trial route on the map in the office, on account of the scarcity of information given on it, in which case the sketching-in of the trial route must be done when on the ground, and may even be revised several times before the exploration work is complete.

Again, the most general case of a reconnaissance survey is that through an uninhabited district, which may require to be crossed in order to reach rich agricultural or mining districts beyond or to make a more direct route to other railway extensions. Here no governing points will exist except those provided by the drainage system and other topography. Consequently the study of the topography in order to find the most suitable route requires a much more careful examination and comprehensive grasp of the nature of the country, as a whole, than in any of the previous cases.

Reconnaissance Survey for Route.—The main object of a

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reconnaissance survey for route is to ascertain on a map, or on the ground, or on both, the approximate position of the trial route to be followed by the traverse line of the preliminary party when making the detailed survey. The above information may be conveyed to the chief of this survey party in various ways. If good maps exist he will be armed with one of these with the route carefully sketched on it. In addition to this, sketches to a larger scale will be given of the governing points and other important points which may be ill-defined on the plan. Again, if no plan exists, rough sketch-plans with the route marked on them are prepared on reconnaissance and given to the chief of party.

Another object of the reconnaissance survey is to select camping sites and the means of transport to such sites.

The objects of a reconnaissance survey when expressed in this way may seem small, but when it is realized that the engineer is investigating properties of a large area and using his own judgment to eliminate all but a narrow strip of it, it will not be difficult to grasp the far-reaching effect of this work and the need for its being well done.

Types of Reconnaissance Surveys.—Many types of reconnaissance surveys could be defined, but dealing with the matter on broader and more general lines, reconnaissance may be divided into two operations :—

The first, a general investigation of the whole stretch of country between two governing points, to enable the engineer to become familiar with the broad features of the country and to decide where one or two trial routes should be made.

The second, a more detailed investigation to define more clearly the actual line which the preliminary survey party shall follow for their traverse. This is usually made a few miles ahead of the survey party each day, and the marks defining its direction on the ground can easily be pointed out by the chief of the party to his assistant or the transitman.

First Reconnaissance.—This can be omitted where sufficiently good maps of the country exist, to enable the approximate position of the trial routes to be located on them in the office. If an engineer inexperienced in this class of work is suddenly called upon to carry out the reconnaissance of a hilly or semi-mountainous district between two governing points, say 60 miles apart, he will at once realize that his task is no easy one. But when, after having taken up his position at his starting point with field glasses and hand-level or clinometer and decided upon one of several possible directions to follow, he moves forward in that direction and again observes that part of his field of vision at closer range, his mind is suddenly awakened to the fact that

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the deceptiveness of appearances alone will make his task a very onerous one. Thus, shadows in a solid range of hills may suggest convenient valleys and mountain passes and, on the other hand, hills overlapping each other may offer the appearance of a solid range of hills, whereas when approached may be found to have promising valleys between them. Also a promising valley, even when observed from closer quarters, may not reveal the fact that it is a *cul-de-sac*, to emerge from which the difficulties are so great that this trial route may need abandoning entirely. Again, the number of possible routes radiating from each observation point, which the inexperienced explorer may be unable to reduce without a closer investigation of each individual one, may be so great that the task of the surveyor will seem endless. A little experience in this class of work will enable a number of these routes to be eliminated in the early stages. Also a knowledge of some system of dealing with these problems will enable the surveyor to set about his work in a more decided, and at the same time exhaustive manner.

The Drainage System as a Guide to Topography.—A complete study of the drainage system of the district will supply very convincing evidence of many of the topographical features such as ridges and valleys. Many important American engineers have pointed out the importance of studying the drainage system. On the first reconnaissance a rough sketch plan of the drainage system should be prepared showing main rivers and their tributaries and as many of the subtributaries as possible. This can be done on squared paper. With such a plan, supplemented by a few levels, much can be accomplished towards sketching in approximate contour lines, and more especially by one having a fair knowledge of geology and its relation to topography. The topographical forms characteristic of each district have their history and origin secreted almost invariably in the geological strata beneath them. Again, valleys which have one side steep while the opposite side slopes very gradually are revealed by a study of the drainage system. Thus the slope of the side of the valley on the right bank of the river in Fig. 1 is obviously steeper than the opposite side where the tributaries are much longer. The slopes of the sides of a ridge can be studied in just the same way by a study of the drainage line on each side of it. Again, the nature and gradient of a valley bottom can be predicted very accurately from the form of the river. Fig. 2 shows a very sluggish river meandering through very level country, whilst a straight river suggests a rapid flow and therefore a steep gradient.

To the engineer who has studied these things, all these and many other facts are so familiar that they become a second nature.

First Trial Direction of Route.—In exploring between governing points it is natural to follow the most direct line, and if possible a

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straight line joining these points, in the hope of finding a likely location there. Still, with the chance of a better location being found

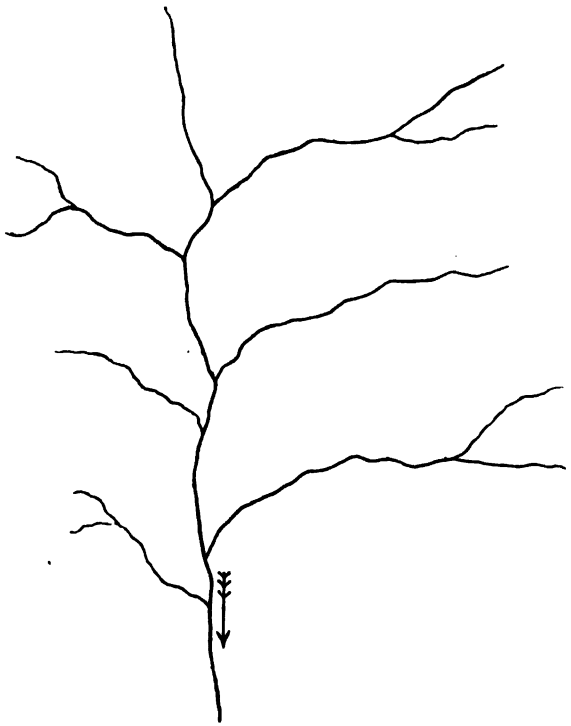

FIG. 1.

elsewhere, it is particularly important that much of the area on each side of the direct line between these tying points should be examined.

There is no worse habit for an engineer to practise than that of

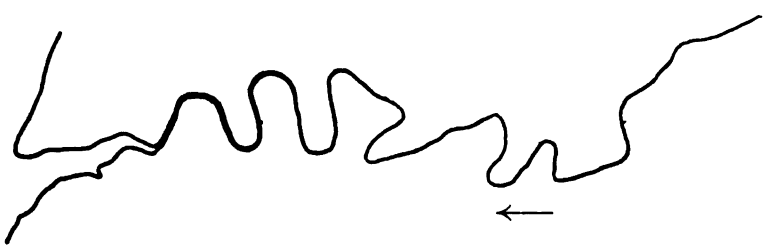

FIG. 2.

assuming the route he has selected is the best without sufficiently investigating the possibilities of all other routes. To condemn a route

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as not feasible should only be done after careful investigation has proved it to be so. Because even if the route is obviously a difficult one, the other more promising routes may subsequently prove impossible and it may then be necessary to continue the condemned route.

Modes of Conveyance.—In his study of the broad features of the country and also the various roads and trails, the locating engineer will be influenced by the distance between the governing points and to some extent the nature of the country and maps available, as to what mode of conveyance he will use. If the governing points are not more than 10 miles apart and the maps on which he has located a trial line good, he may find it most convenient to travel on foot on account of the deviations from that trial line being very small. But if the trial line connects more distant governing points, or if inferior maps render this trial line of very doubtful value, a much broader belt of country will need investigating, and travelling on foot will be much too slow, and will also deprive the locating engineer of that broader view of the features of the country which are so essential when conducting reconnaissance. Travelling on horseback or in a light cart is more satisfactory for this work; occasionally where roads exist a bicycle with cyclometer has proved very serviceable. Of course, occasionally, it occurs that even a horse cannot get through the undergrowth, and then there is no alternative but to make the exploration on foot.

The Instruments most Commonly Used for Reconnaissance

The prismatic compass, the hand-level, or better a clinometer which can be used as a hand-level, such as an Abney level, the field glasses, and the aneroid barometer are principally used. A stadia theodolite may be used to make a rapid survey of the country where no maps are available, but in dense forest, as is found in the tropics, this is useless and the barometer is essential to obtain levels. This would mean both an aneroid with the explorer and a mercurial at some central station.

Although many engineers believe implicitly in the use of the barometer for levelling purposes on most exploration work, whether in the open or in dense forest, its advantages are very doubtful in the open where stadia surveys can be made so rapidly and afford as high a degree of accuracy in the levels as can be obtained in the horizontal measurements. (The method of using the barometer will be dealt with later when discussing exploration in tropical forests.)

The Prismatic Compass.—The properties of the prismatic compass are given in any book on Plane Surveying; but its advantages for this class of work are: its compactness, the ease with which the bearings of objects sighted upon can be taken to 30 minutes of arc.

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This is due to the magnifying power of the prism. The graduations usually give whole circle bearings from 0° to 360°.

The Hand-Level.—This is usually about 5 inches long, though some can be pulled out telescopically to a length of 6 or 7 inches, which is an advantage for steadying the bubble. The hand-level, which should be a good one and tested for accuracy, is useful for sighting objects and parts of the terrain at the same level as the explorer.

The Clinometer.—This is somewhat similar to the hand-level with a vertical graduated arc attached, the bubble tube being fixed to the pointer and free to move to any angular reading on the arc. When set at zero it is parallel to the telescope and can then be used as a hand-level.

The Tacheometer or Stadia Transit.—This instrument, which has been much used in India in the past, is proving its value by the more frequent use to which it is now put. It is much used in North America for railway work. It is most useful where no maps of the district exist or where the maps that do exist contain little or no natural topography. It is possible sometimes with one of these instruments and four or five stadia rod men to make a rapid topographical survey of a belt of country about 2000 feet wide and extending 5 or 6 miles in one day. Of course, this survey does not contain small detail, but a good general sketch of the country, in which all the prominent features are shown, the levels being of about the same degree of accuracy as the horizontal measurements.

Methods of Measuring Distance.—The matter of measuring distance on reconnaissance is a difficult one except in the case of stadia work, or where a bicycle with attached cyclometer can be used. Odometers (an odometer is a wheel which records the distance covered by it as it is wheeled forward) and pedometers (these give the number of paces taken) are recommended by some, but the former are unsatisfactory as they need too much attention. Probably the most suitable method of measuring distance is by timing. A man walking long distances keeps to a regular stride, except occasionally in difficult ground and up or down hill. The same applies to a horse when ridden or driven long distances. Its average speeds for walking, trotting, and galloping can all be tested and will remain wonderfully constant. Thus the distance from one point to the next can be determined by observing the time of passing each one, and paying due attention to the mode of travelling from one to the other, and modifying the result if the country has been very hilly or difficult to traverse. For this purpose field notes are frequently taken with a time column along one edge of the page, and the central space used for rough sketches showing the objects

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passed at the times on the same lines. There is no doubt the notes can be taken quickly, though the descriptions at times may be long, but these notes are afterwards used for plotting the observed detail upon the map of the country.

Forms of Field Notes.—Probably the best method of obtaining notes is to plot them directly on to portions of the map of the country, which is drawn to a scale of 1 inch to the mile, the map being covered with 1-inch squares representing sections. Maps to this scale can usually be obtained. And even if maps of the district are non-existent it is probable that field notes on this inch-square paper are most serviceable, as they enable the explorer to place the objects observed more correctly in their relative position, and they need not be any source of delay. The method will be more clearly understood by studying Fig. 3, which represents a page from a field note-book used on reconnaissance. This note-book was made of sections of the only available maps of the country. These maps gave the section lines and the rivers. The maps were cut into strips, 6 sections long by 3 wide, and these strips were pasted into the left-hand pages of the note-book.

Distances were measured by a pedometer and also by time. These methods proved of about equal value, and at times equally useless, as it was found when traversing difficult ground the average length of the stride was not maintained and also the time per mile was greater. However, checking on the section and quarter-section corners prevented any accumulation of error in distance.

In Fig. 3, starting from section corner 20, 21, 28, 29 (the latter two only appear on figure), and moving north by prismatic compass a distance of about three-quarters of a mile, the line of flood level cuts the path taken and verifies a probable position of this line as observed a quarter of a mile back. This enables a few points on the flood-level line to be marked on. Also a sand hill was observed about half a mile to the left.

The flood line continued along the path to the next section corner (29, 28, 33, 32), and at this point the sand hill was estimated to be about a quarter of a mile to the left, whilst on the right a portion of the river valley is occupied by a swamp.

Having sketched in these, including the flood-level line up to this point, and also the approximate edge of the sand hill, the direction is continued north, and more points are located in the flood line and also the sand hill. A trail running east and west is crossed at section corner (32, 33, 4, 5), here the aneroid reading gave reduced level 210·0, here also the trail runs north following the section line fairly closely. Having continued north for about half a mile, the trail crosses Gander Creek. A level well-drained sandy prairie close to good running water on the south side of the creek suggested a suitable camping

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site. On the right the flood level approached to within 500 yards. Continuing to section corner (5, 4, 9, 8), a high mountain is located with its base about half a mile to the left. The trail now takes a

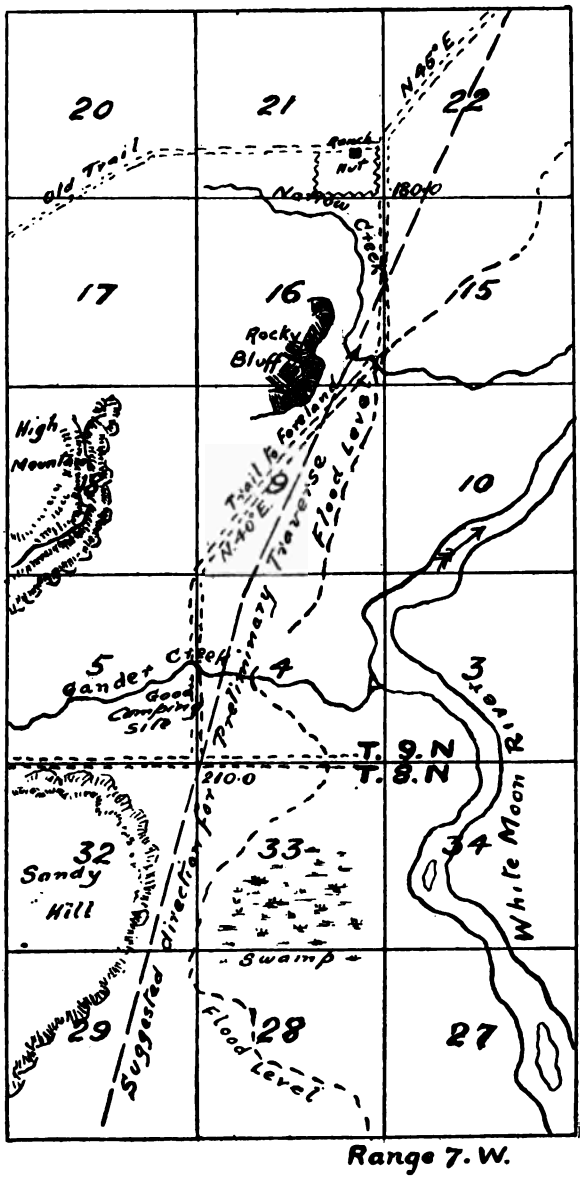

FIG. 3.

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direction about N. 40° E., the direct line north running into hilly country. Following this till it passes close under a rocky bluff, the point where it crosses the section line is estimated, and then a direction due east taken till section corner (9, 10, 15, 16) is found.

Moving north from this point about 200 yards Narrow Creek Stream is crossed and flood level located about 100 yards further. Having thus continued till this page of notes was complete, the points marking the suggested position of the preliminary survey are connected up as shown in Fig. 3. Here the governing points are (*a*) between the sand hill and the flood-level line, and (*b*) between the rocky bluff and the flood-level line.

Notes of this kind can be made rapidly and many miles covered in the day. But if it should be necessary to embrace a wider strip of country this will probably necessitate a parallel trip being made some days later, as it is usual for locating engineers to return over the same route they have followed on their outward journey, as the topographical features of the country present a rather different appearance when viewed from the opposite direction, and much further detail can be noted and that already noted modified.

Selecting Areas and Routes.—Much of the country between governing points can be covered on reconnaissance, but it is necessary to make a sketch map of the whole belt covered. In difficult country a study of those points which topographically govern should be the principal object of the first exploration. Then having found the limits of their controlling effects and located their positions, the engineer is in a position to decide if any of these governing points should be abandoned, and how best to connect those still retained. It is much better not to abandon any controlling points than to do so without thoroughly investigating the matter, as a route once condemned will not be further investigated during the period of conducting the survey. If the previous areas of country surveyed suggest that a ·6 per cent. gradient or about 1 in 166 shall be the ruling [1] gradient if possible, and that the maximum curvature allowed will be that given by a 4° or say 22 chains (66 feet) radius curve, and it is found that by making a fairly direct line towards the governing point renders the gradient steeper than the ruling gradient, a further attempt may be made for approaching this governing point by means of development.[2] Yet it generally happens that where development is needed the chances of accomplishing it satisfactorily, and at the same time conforming to such strict curvature requirements would be small. But this is no reason for failing to thoroughly investigate the possibilities of such a route, which,

[1] For ruling gradient see p. 41.
[2] For development see p. 18.

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if finally adopted, may prove to be that which combines best with the most suitable location beyond the governing point.

Precautions in Sighting on Governing Points.—When fixing these topographical governing points on the sketch plan or map, rapid work can be done if compass bearings to them can be taken from known points on the map, such as section corners if these are given and can be found on the ground. In making compass observations of this kind the actual object which can be sighted on may not be the governing point but near to it. Such objects as mountain passes are seen very differently from different points, and it is frequently found that the second observation is on a point some distance away from that first observed on account of the parallactic displacement of the sides of the mountain pass. For this reason it is well to check the position by making one or two more compass observations, or as the explorer will need to pass through this governing point when investigating its possibilities, a bearing and a distance will fix its position, the distance probably being determined by time. But the most satisfactory method of locating on the map such governing points is to observe some definite point, if it is possible to see one, such as a pointed mountain top or an isolated tree, which is near to the point needed, and later to fix the true position by a bearing and distance from the tree or hill top.

Routes and their Limitations

It is essential that the locating engineer should have been given with his other instructions some idea of the maximum cost per mile which will be permissible. It is an easy task for an engineer to locate a first-class line if he is almost unlimited in the amount to be expended per mile, but few railways are constructed under those conditions. The constant demand for the best road at a moderate cost per mile has rendered the work of the locating railway engineer a high art. Thus with a moderate though reasonable sum per mile at his disposal the explorer soon finds it easy to eliminate certain routes. It is impossible to construct a line economically without following the natural topography. Take the instance of a river valley where the average fall in the river is 1 in 100. It would be ridiculous to attempt to construct a line to a ·6 per cent. ruling gradient in this valley, as not only must the fall of a valley be less than the ruling gradient, but frequently it requires to be considerably less to enable the line to emerge economically from the top of the valley. Again, what seems an easy valley on account of the gentleness of its fall, may prove expensive because of the windings of the river, making many bridges necessary, in order to adhere to a straight line with flat curves.

Valley Route or Ridge Route.—In dealing with the problem of

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selecting a route by studying a map of the drainage system of the district, the most natural question which arises is, Which is preferable, a valley route or a ridge route ? A valley route almost always appears to be the easier, with its long lengths of straight easy curves, and little or no rock excavation. Whilst the ridge route will probably run for most of its length in rock and have a much more winding route with sharper curves.

On the other hand, the valley route has to keep its embankments above high water at flood times, and also guard against undermining by the river at such times. Then all tributaries have to be crossed, sometimes demanding expensive bridges, and if the main river needs crossing the cost per mile is at once increased considerably, while the ridge route will need no bridges and the formation or grade level can be kept low because no water is likely to collect near.

Thus it would appear that the ridge route might cost least to construct at the sacrifice of its alignment, which would be inferior to that of the valley route. But it is less likely to pass through towns than the valley route, and even where towns exist near it all freight has to be hauled uphill to the railway. There are many cases of competition between roads following these two alternative routes, and in no case where the conditions are otherwise equal has that following the ridge succeeded in getting even an equal share of the traffic. Thus it is usual to give a preference to valley routes in making explorations.

It often happens that the direction to be followed by the railway requires the main valleys to be crossed nearly at right angles. These cross-ridge routes are very difficult, as not only is the valley crossing a very wide and deep one, but the ridges on each side are at some of the highest elevations in the district, necessitating very heavy working surmounting the ridge in such a short distance. No doubt, at times, tributary valleys may be found to help to solve this problem, but it is hardly to be expected that they will be found conveniently in each successive valley. It is never expected that a cross-ridge railway can be constructed without a greater outlay than usual.

On first-class lines it sometimes happens that a short tunnel may be substituted for a portion of line much "developed" in surmounting a ridge and thereby shorten the line and improve the gradients with no more cost than that of constructing the "developed" line. This generally means finding a point where the mountain ridge is narrow and steep if possible on both sides.

Development

Development in railway location is the term used for lengthening the line in order to reduce the gradient. For instance, the Canadian Pacific Railway from Calgary to Field passes through the Kicking

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Horse Pass where a gradient of 4·4 per cent. was operating some years ago. This has since then been improved, the gradient being reduced to a 2·2 per cent. one, by this process of "development," which in this case meant doubling the length of the line. Development may be effected in various ways, such as by switchbacks, zigzags, loops, spirals, and other curves, but the conditions under which this can be easily done by no means occur everywhere.

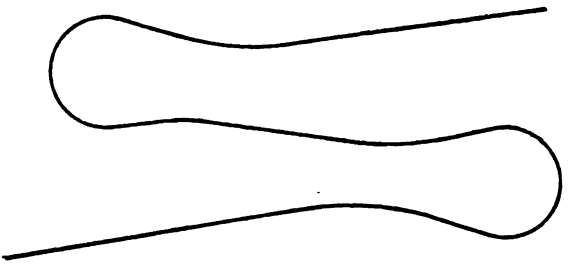

FIG. 4.

Zigzags and Switchbacks.—A zigzag is the path a horse takes when hauling a heavy load up a hill, the motion being from side to side of the road. In railway work (see Fig. 4) this side to side movement requires to be long so as to reduce the number of turning curves, each one of which for a portion of its path is travelling directly uphill, and in order to preserve the gradient, or indeed flatten it, because of the compensation for the curve, rather heavier earthworks will be

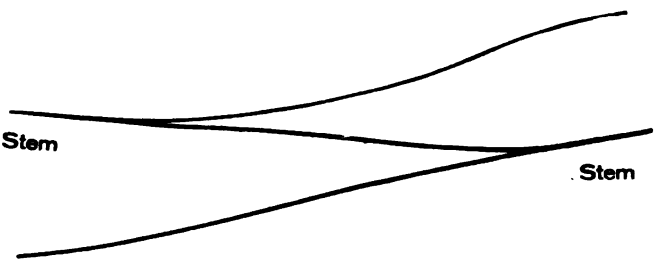

FIG. 5.

needed at these points. Although zigzags may be successfully constructed up the face of a broad slope, whose direct ascent is much steeper than the ruling gradient, yet as this slope gets steeper the turning curves become more difficult and expensive to construct, and eventually such construction becomes "out of the question." Then it may be necessary to resort to switchbacks (see Fig. 5), as they do not need any turning curves, their places being taken by stems into which

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the train is hauled before being pushed backwards up the next slope or vice versa. These stems are constructed with the gradient increasing up to the end in a long vertical curve so as to bring the train to rest at the upper end of the curve and help it to start in the opposite direction with the thus gained potential energy. Where the vertical curve has been properly designed for trains moving at the average speed up-grade, the train will be brought to rest gradually without shutting off steam and upon immediately reversing the gear will resume its ascent up the next incline with the initial velocity provided by its increased elevation in the vertical curve. Again, descending trains, by being brought to rest at each stem, are kept easily under control even on a steep down-grade. However, an objection which cannot be overlooked is that on alternate inclines the engine has to push the train. With the engine thus running backwards the motion, especially on curves, is not good. The switchback is an excellent arrangement for ascending great heights where the traffic is not heavy.

Loops or Spirals.—A loop, as the name implies, is the looping or doubling back of a line in plan so that it closes on itself. But the two

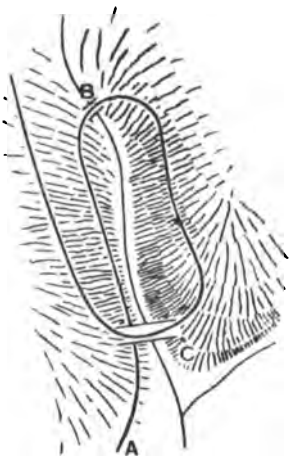

FIG. 6.—Bridge spiral.

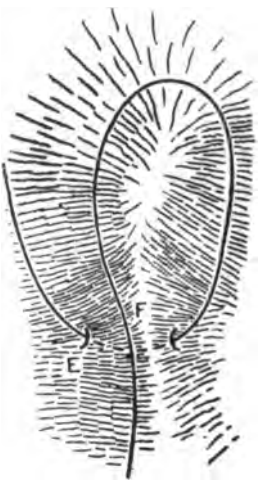

FIG. 7.—Tunnel spiral.

portions of the line where they cross each other are widely apart in elevation, one being at a much higher level than the other. This looping can be accomplished in one of two ways, either by bridging over one portion of the line, in which case it is called a "bridge spiral," or by tunnelling under one portion of the line, thus forming a "tunnel spiral" (see Figs. 6 and 7).

In the case of the bridge spiral (Fig. 6) the line from A to B is

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ascending a valley which is steeper than the "ruling" gradient. Consequently upon reaching B it is getting near to the level of the bed of the river and has no alternative but to bend back and continue its up-gradient on the other side of the valley to C. Here the valley, being steep and narrow, is crossed by a bridge at a level high above the first portion of the line between A and B, and thus enables the line to continue farther up valley. In the case of the tunnel spiral (Fig. 7) a similar difficulty had arisen, but this time by tunnelling through the mountain ridge separating the two valleys, at E, a point where the ridge was steepest on both sides and consequently the distance through short, the line was enabled to get into the adjoining valley, and by skirting round a ridge peak to return into the same valley at a much higher level at F. In this way it is often possible to keep the gradients within the ruling gradient, and also to obviate the difficulty of driving the trains backward, as is necessary in a switchback.

There are also many cases of tunnels nearly forming a complete loop in the mountain before emerging at a much higher altitude. The gaining of altitude in this way can sometimes be effected by turning up a tributary valley for some distance and then crossing over and returning on its opposite slope, thus enabling the line when returned to the main valley to be at a much higher altitude than on leaving it.

Second Reconnaissance

This, which is for detail, should be conducted by the engineer in charge of the preliminary survey party, which is engaged upon the preparation of the topographical plan upon which the proposed line will be "located." He should keep his reconnaissance work well in advance (at least 3 or 4 miles) of the preliminary party so that there need be no delay caused to this party, and also to enable him before finally selecting the line for the traverse to know what is ahead of him and thus have a broader and more general knowledge of the country.

It is always advisable that the preliminary survey party should, in addition to a chief of party, have an assistant chief, whose business will be to entirely look after the party and keep the transitman informed of the direction the line is to take. The assistant obtains the information concerning the direction of route in various ways. Some chiefs, before starting on their day's reconnaissance work, show the assistant over 2 or 3 miles of route, and later in the day, when returning over his reconnaissance route for verification and final selection, will supplement the information given in the morning, if the party is in need of more. But a method which gives most satisfactory results, especially in country where plenty of distinctive topographical features are present, is that of making rough sketches on square-ruled paper. These, though rough,

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if they contain the general topographical features to be encountered and any small detail on the direct line, and if they are also carefully numbered, should enable the assistant to find his way tolerably well. But at times the featureless condition of the country would render even these difficult to follow; in such cases it is advisable to define the line on the ground by blazing trees or tying strips of coloured canvas or other material to trees or undergrowth, or in open country by sticking vertically in the ground branches of trees or rough sticks at intervals along the route.

Although the first reconnaissance has afforded a route to be followed by the locating engineer, it is only a very general one, and though it narrows down the strip of country requiring investigating, there is much still needed to make the reconnaissance work complete. Thus in the first reconnaissance the general drainage system and some of the tributary systems will have been roughly mapped, but the rough work of investigating the source of every small stream and whither it is flowing, the most suitable crossings of the larger of these, and all minor topographical features falls to the lot of the chief of party.

The Objects of Reconnaissance during Preliminary Survey.—But it is not only for the purpose of selecting what seems the best route and of conveying that information to the assistant chief that the locating engineer spends nearly all his time exploring the country in advance of his party. In so doing he is getting a more intimate mental grasp of the country, and its various peculiarities of detail. Thus he is enabled to decide what other routes could be tried, if it is found necessary to change the route, or even to have two trial routes surveyed at the same time on account of their seeming of almost equal merit.

Indeed his reconnaissance experience is invaluable to the chief of party for all the subsequent work of his party up to the moment the final location is fixed on the ground; as at all times when dealing with the various phases of the work, he is able to recall to his mind the further possibilities of the country, and frequently can improve his line by making further short deviation surveys, the advantages of which were not observed in the first instance.

Starting the Survey.—When the preliminary survey party arrives at the site of the first camp, the chief of party at once must give his attention to reconnaissance, so that there shall be no delay in starting the traverse when the party is all ready and in proper shape.

The reduced level of the starting point is probably known, but if not it may be necessary to assume a value for it. A number of readings may be taken with a mercury barometer each day, so as to arrive at an approximate value for it, and an aneroid may be carried on

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the reconnaissance, and readings taken on the tops of ridges and bottoms of valleys. The method of using the aneroid for this work is given on p. 28. A hand-level, or clinometer, which can be used as a hand-level, is also very necessary.

Advancing on a Bearing of Direction Selected.—In starting to ride over the strip of country marked out by the first reconnaissance, the chief of party will endeavour to keep a direct course on what seems to be the most suitable line for the traverse. To follow the direction selected by prismatic compass is very difficult, but need only be resorted to where it is impossible to see a good distance ahead. Where the country is fairly open two or three objects, *e.g.* a group of trees, or mass of rocks, at a considerable distance apart on the line of direction taken, can be easily kept in line with each other, and a fresh one selected as each one is passed. In this way it is possible to follow a very direct line. On the summits of ridges it is sometimes difficult to extend accurately the direction on account of the nearness of objects sighted upon. In this case, probably the simplest cure is to revert to the prismatic compass, taking a sight with it before surmounting the ridge and extending the bearing thus taken by sighting beyond from the top of the ridge. Fresh objects will then be selected on this line for sighting on in order to keep in line. The chief of party may find it advisable to change the direction when he gets a clear view beyond the ridge, but in that case the turning point would be on the ridge. Thus by accurately keeping on the direct line, the estimation of distance traversed is much more reliable, and the points to be recognized by the survey party following can be more quickly found.

Forms of Notes as a Guide to the Transit-party.—In addition to the approximate bearing and the distance to each turning point, which the chief of party sketches on a loose leaflet, each prominent object passed is also shown, and the distance to it, so that the transit-man's line, if following the bearing more accurately, may not diverge from the intended line by any great amount before checking on one of these points. It is also of advantage on occasions to make sketches of distance views like skeleton photographs or horizon sketches. Thus sketch, Fig. 8, shows the low point on the ridge D, to which the party may look on reaching C to get the new direction. A further check on this direction is provided by the angle subtended by the points D and X (a prominent rock point on the right), at the observer at C. This generally removes any doubt about which low point on the ridge in view of the observer was intended. On the plan sketch which the view sketch accompanies, the bearings of the lines CD, CX, etc., are all given. If the country is lacking in clearly-defined features, the horizon is often the only part which shows out distinctly. In such a case the

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horizon point is very important, being much more easily detected than strips of white or red canvas tied to bushes or trees, even though their positions may be clearly shown on the sketch plan.

The following routine will probably be found useful on reconnaissance:—

One page in the field note-book, say the right-hand one, should be given up to recording the following in columns: the time of arriving at and also departing from any observation point; the estimated rate of travelling in miles per hour. A column can be left for distance which can be filled in afterwards. The reading of the aneroid barometer. (This should be taken immediately upon arriving at the observation point; another should be taken immediately before starting forward again.) A column is also left for entering the elevation. (This is

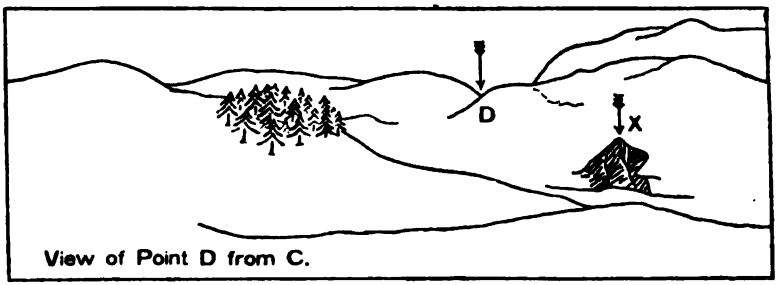

FIG. 8.

either done at the time or after corrections have been applied to the aneroid reading.) The compass readings are also recorded in another column. If possible, a space should be left for observations.

It is generally found that a ruled strip at the top of the page is sufficient for entering these observations, thus leaving most of the page plain on which to make sketch plans. Many more plain pages are required than recording pages. With this arrangement most of these pages can be used for sketches, and where observations are recorded on any page the column lines can be extended to the bottom of the page, thus providing space for a good many records. Whether measuring distance by pedometer, cyclometer, or time, it is advisable to record the time of arriving at an observation point as well as that of departing from it; also the pedometer or cyclometer reading must be recorded immediately afterwards. If these columns are arranged in this order, little chance of omission is likely, as each entry on the same line will be made in its own column in the order on the page. The barometer reading must be taken next and another one just before the time reading when departing from the station. If the halt at the station has been short, only a few minutes, the two readings of the aneroid may

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vary so little that the average of the two may be used in determining the elevation, but if the difference in elevation from the two readings is considerable the elevation is determined from the first reading, the second reading being used in determining the difference in height to the next observation point. If the weather should become unsettled, sudden changes in the barometer may occur. When such is the case great care must be taken to find out how much variation is due to weather changes, and how much to change in elevation.

The method usually adopted for this purpose is to compare the aneroid readings at the various times with readings taken upon a mercurial barometer at equal intervals of time throughout the day at some central station or headquarter office. This latter will show any variations due to weather changes which will be assumed to be the same at the observation point. Thus it is usual to correct all aneroid readings by comparing them with the mercurial readings when the observer gets back to the office.

As often in the field as it may be convenient to make use of the aneroid elevations for the purpose of making sketches showing elevations and contours, this can be done, and if any correction in an elevation is afterwards found necessary it can be made upon the sketch in the office.

The frequency of making observations depends upon the nature of the country being traversed. If comparatively flat, it may be unnecessary to do so at intervals less than a mile. But even in easy country, it is generally advisable to make observations every mile. In more broken country it will be found necessary to take them more frequently.

The taking and sketching of cross-sections at observation points are advocated by many, but the results obtained from taking a cross-section can easily be entered upon a topographical sketch and facilitate the addition of more detail observed at the same time, which could not be shown on a cross-section. The clinometer enables cross-section details to be easily obtained, especially upon side-sloping ground.

When first running between governing points as direct a path as possible should be taken, but at the same time one which could be followed for the railway. On this occasion it is necessary to note all rivers, ridges of hills, etc., crossed by the path taken, and to take the bearings of the directions of the ridges of hills and of the rivers. On a subsequent occasion, when passing through this area, the rivers and ridges must be followed for some distance on each side of the line and mapped, and main rivers should be followed to their sources.

Where it is decided to try and select a route along the side of a valley to its summit, the work is generally accomplished by starting from the summit and working back to the line. But on reconnaissance

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this can usually be avoided by sighting forward with the clinometer and so selecting at the entrance to the valley how high upon the side of the valley the line must run in order that it may reach the summit on a gradient no steeper than the ruling gradient. If the aneroid is read at frequent intervals, it will check the accuracy of the clinometer and enable the route to be corrected if necessary. If a route is selected in this way by the clinometer it will be found to work out more satisfactorily on the ground, as the path it follows is more winding than the direct line of sight of the clinometer.

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CHAPTER III

RECONNAISSANCE IN TROPICAL COUNTRIES

In the dense forests which cover much of the tropics exploration and reconnaissance work has to be conducted on quite different lines from those already described. The impossibility of getting a general view of the country owing to the enormous growth of these forests, renders it necessary to have recourse to some other method to obtain the information needed about the country. But although the first exploration, as previously described, of an open piece of country may be so far reaching that it enables most of the country to be excluded from subsequent investigation and probably the best route to be selected, yet, the fact that in many of the dense tropical forests, trails or paths and trade routes taken by the natives lie near the line most suitable for a railway, enables adherence to these routes often to lead to useful results. It is a remarkable thing that the natives, in moving to and fro through these tropical forests, should have selected for their paths almost the most direct and easy route to travel by. Still, this must be largely due to their having been travelled over probably for many generations, and that they were evolved out of much less satisfactory routes which were gradually shortened and eased through the natives discovering better ways by improving the gradient or cutting off corners. However, no matter how near these paths may eventually prove to be to the best position for the railway, the need for conducting such exploration surveys economically makes it obligatory on the surveyor to make use of these paths for the purpose of deriving more information about the nature of the country topographically. Also, the paths will naturally lead the way to finding out other information so essential on an exploration survey, namely, the sizes of towns and villages and their populations, what amount and kind of agriculture is carried on, any other industries of the natives and the amount of trade done, and the traffic on the trade routes. Of course there are tropical forests, notably in South America, which even lack paths in many parts, where few or no inhabitants are to be found, and in such the problem is a much more difficult one. To overcome this difficulty it has sometimes been decided to cut

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a path through the forest in a straight line for many miles at considerable expense. But this chance effort to get a base or traverse from which to survey for the railway cannot but fail, as it may cut through high mountains and deep valleys which could have been avoided by making choice of the direction of the traverse as the line was being cut forward. The only possible advantage in thus cutting out a straight line lies in the fact that it makes the shortest line between the places to be connected by rail, and if it is possible to build the railway near that line it should be built there, provided it is not considered worth while to pass near other towns on the route. But a traverse line must be cut through in order to prepare even a rough plan of the district, and it may as well be done with a view to getting good gradients. It has at times been found convenient, when small hills are encountered in carrying a traverse line forward, not to turn aside along one slope of the hill, but to carry the line over the top in order that the topographical detail of the country on each side may be investigated with a view to selecting the best side of the hill for the railway.

First Exploration or Reconnaissance

Before attempting to cut any portion of a traverse line, several miles must be covered on foot or horseback in the direction of the direct route to discover what obstacles may be encountered and afterwards, if necessary, the country on each side of this direct line should be similarly investigated. It is on these trips that particulars of the nature and products of the country are collected, as well as some general topographical and, if possible, geological information.

The only instruments used on these trips are the aneroid barometer and the compass, the distance being determined by timing.

In riding through the forest, if even winding paths can be followed, it may be possible to take short sights from each of the bends of the path to the following one, and obtain its magnetic bearing with the prismatic compass, determining the distance by the time taken in moving from one to the next. The aneroid readings must also be taken at any of these bends where changes of natural gradient are evident. However, where the distance from one bend to the next is very short, in some cases only a few yards, the method would become too tedious and the linear measurements unsatisfactory. In such a case a surveyor accustomed to this kind of work will obtain data for roughly plotting the route he has taken by riding forward at a uniform rate and taking readings of the compass, watch and aneroid every few minutes.

The Aneroid Barometer for Measuring Elevations.—The modern aneroid gives such excellent results when properly handled that many engineers, who have been in the habit of using them in the past,

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continue to do so now even on occasions when they might be dispensed with, on account of the closeness to the true values of the results obtained. On account of its convenient size the 2½-inch or 3-inch instrument is much used on survey work, but it suffers from the tendency to lag. This lagging is due to having too small a vacuum chamber, which cannot be avoided in so small an instrument. On this account the 5-inch instrument is very strongly recommended, but being so much larger and more cumbersome it is sometimes inconvenient to carry on reconnaissance. Mountain explorers know well this tendency to lag in aneroids, as it is most noticeable where great changes in elevation are being observed, the difference in height recorded at ascent at the outward trip being greater than at descent when returning. Still, on railway reconnaissance this is not a very serious source of error, as the variations in height are not so great. Although these instruments are said to read to one foot, this simply means that it is possible to observe a difference of a few feet in elevation recorded upon the instrument. In order to get close results with an aneroid the ground requires to be traversed in both directions at least two or three times and the average taken. This will sometimes reveal differences of 10 per cent. to 15 per cent. in small elevations, between individual readings taken when going in the same direction over the same piece of country, and 5 per cent. to 10 per cent. in the case of much greater elevations. However, on railway reconnaissance it is rarely possible to test the levels more than on the outgoing and return journeys over the same strip of country. But as this will generally give levels within 10 feet of their true value, if the instrument is a good one and known, and also handled carefully, it is generally sufficient for the purpose.

The aneroid should be used on reconnaissance work in conjunction with a mercury barometer. This latter must be established at some standard or central station, and readings of it taken frequently throughout the day at regular intervals, so that any change in the atmosphere recorded by it may be taken account of when considering the readings taken from the aneroid at various points on the route. When the readings of the mercurial barometer are being taken, those of the attached thermometer must also be recorded. It is also usual to record the readings of another "outside" thermometer which is detached from the structure or house in which the mercurial barometer is established, so that it may give the actual temperature of the air and its changes during the full period of observation. Thus it may be the duty of the man located at headquarters or wherever the standard station may be, to take readings every 10 or 15 minutes from each of these instruments, and record them in a table with the time of taking each group of them. It then becomes only necessary to interpolate

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values at times between these to compare with the readings taken on the aneroid at specific points on the exploration trip.

The usual method of recording the readings at the mercurial station is shown in Table I.—

TABLE I.

Time of taking reading.	Temperature of external air.	Temperature of internal air at barometer.	Barometer reading.	Correction of barometer reading for temperature reduced to 32°.	Corrected reading of barometer.
10 A.M.	58° F.	66° F.	30·095	—·101	29·994
10.10 A.M.	58° F.	66·5° F.	30·096	—·102	29·994
10.20 A.M.	58·5° F.	67° F.	30·099	—·104	29·995
10.30 A.M.	59° F.	67° F.	30·105	—·104	30·001

The first four columns contain the actual data entered by the observer, whilst the fifth column contains the amounts by which the barometer readings in the fourth column require correcting, so that results may be compared at 32° Fahr. The sixth column contains the corrected readings.

Various arrangements of recording tables have been tried when using the aneroid barometer in the field, of which the following is quite as useful as any :—

TABLE II.

1. Time of taking reading.	2. Temperature.	3. Aneroid reading.	4. Corrected aneroid reading.	5. Corrected mercury reading interpolated if necessary.	6. Elevation from aneroid uncorrected for temperature
10 A.M.	59° F.	30·073	— [diff. —·079]	29·994	5 feet
10.15 A.M.	57° F.	30·040	29·961	29·994	35 ,,
10.28 A.M.	54° F.	29·632	29·953	30·000	410 ,,

7. Elevation from mercury reading uncorrected for temperature.	8. Difference of elevation.	9. External temperature interpolated if necessary.	10 Correction for temperature.	11. Corrected difference of elevation.	12. Place.
5 feet	0 feet	58°	0 feet	0 feet	Headquarters
5 ,,	30 ,,	58°	+ ·6 ,,	30·6 ,,	On road bridge
0 ,,	410 ,,	59°	+7·8 ,,	417·8 ,,	Top of sandy hill

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Explanation of Table II.—Only columns 1, 2, 3, and 12, are actually used in the field, the others being filled in in the office.

The figures in column 4 are determined by the differences in the readings of the aneroid in column 3, and the corrected mercury readings in column 5 (these are entered from column 6 of Table I.). In starting out it is necessary to take the first readings near the mercury barometer at headquarters. Then the difference between the reading of the aneroid and the corrected mercury reading is added to or subtracted from as the case may be, each subsequent aneroid reading to derive the readings in column 4. Thus the corrected mercury reading 29·994, taken at 10 A.M., is deducted from the aneroid reading 30·073, taken at the same time, and the difference ·079 is subtracted from each subsequent aneroid reading, and the results placed in column 4. Column 6 contains the equivalent elevations for the readings in column 4 taken from Table XX. in Appendix I. Similarly, column 7 contains the equivalent elevations for the readings in column 5.

The readings in column 5 derived from Table I. may only be obtained in some cases by interpolation if the times of observation are not the same. Also the temperatures in column 9 may only be derived from Table I. by interpolation. Column 8 contains the differences between the values in columns 6 and 7. Column 10 contains the corrections to be applied to column 8 due to the temperature in columns 2 and 9 by using Table XXI. in Appendix I. Thus for the difference of elevation 410 feet in column 8, the temperatures in columns 2 and 9 are 54° and 59°, whose sum is 113°. In Table XXI. interpolating between readings + ·016 and + ·021 we get + ·019 for 113°. Then 410 × ·019 = 7·8 feet, the correction in column 10. In using the aneroid it is advisable to take a final reading next to the mercury barometer on returning to headquarters, as a further check on the aneroid readings.

The United States Coast and Geodetic Survey carried out most careful and exhaustive investigations on the behaviour of the barometer at different elevations and under different temperature and hygroscopic conditions, and their report has been abstracted in Appendix I., p. 279.

Selecting the Position for the Traverse.—When the country has been explored by riding through, it is possible to decide upon the best position for the traverse of a preliminary survey, which will have conveniently within its reach by cross-sections, any difficult parts of the country where a choice of routes may require careful investigation. Where existing trade routes or wide paths are in existence and convenient for the purpose, this main traverse will be limited to the open space so formed; but where no such conveniences exist, the traverse line must be cut out through the forest, not necessarily in long

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lengths of straight, as it is more important to turn a small angle for the purpose of avoiding any obstruction than to preserve a long length of straight. Thus short lengths of straight in traverses, sometimes only 1 or 2 chains in length, will form as useful a base from which to locate the topography of the country as any long length of straight, which may require many obstructions to be removed at considerable expense. Hardwood trees, sometimes 9 to 12 feet in diameter, in tropical forests, are costly obstructions to remove, and many of them may occur upon the line which might have been avoided by turning it through a small angle.

Theodolite or Surveyor's Compass.—In establishing a traverse or base where it is intended to chop a narrow "clearing" for it through the forest, a theodolite may be used as on any preliminary railway survey. But taking into account the frequent turning points, it may be found too tedious and slow an operation. In that case, as in the case of all surveys of the existing paths or routes through the forest, a surveyor's compass is found much more expeditious, and at the same time sufficiently accurate. It also has the great advantage that any mistake made in a reading does not cause the plotted position of each subsequent line to diverge more and more from its true position, the farther it lies from the point of error. This is what happens when mistakes occur in reading deflection angles by the theodolite.

The Surveyor's Compass.—The surveyor's compass has a dial as large as and sometimes larger than a miner's dial. It is graduated from 0° to 90° both east and west from the north and south line. The "east" and "west" points being interchanged on the dial, so that readings may be taken direct by looking at the dial and needle point. The compass is mounted on a tripod and is fitted with a ball and socket joint, by clamping which, while observing the level bubble or bubbles, a rough levelling of the instrument is performed very quickly. A pair of vertical folding sights (or sometimes a telescope) are used for sighting the line. The miner's dial has also been used successfully for this purpose. It is usually graduated from 0° to 360°.

Although the value of the prismatic compass for some work of this kind cannot be denied, it is found to be unreliable on these long winding traverses needing many readings (probably 200 or 300) every day in a hot damp climate. Moisture condenses on the inside of the glass and the needle dial is liable to stick to it at times and thus give false readings. The needle also takes a long time to settle.

Again, the surveyor's compass is much less liable to be damaged than the theodolite in moving through the forest, and has all the advantages of speed and lightness. The only objection in using a magnetic instrument occurs where local magnetic influences are at

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work, as in the neighbourhood of iron deposits. But even errors from this cause may be discovered and cured by always taking backsights and foresights at each station, the backsight at each station checking the foresight from the previous station. If it is found at any station that they do not agree, that will probably be the first station where local magnetism is felt. At such a point the difficulty may be got over by deducting the obtained bearing of the subsequent line from that of the previous line. In this way the angle between the two lines is obtained, as with a theodolite, as the local magnetic influence affects both readings alike at the same station, and from this the correct magnetic bearing of the subsequent line can be calculated.

Measuring Distances.—Chainage may be taken with the ordinary surveyor's chain, but if so it is preferable to use the heavy iron type to those of light steel, as the latter get damaged in being hauled through the undergrowth. If the chainmen are careful much more satisfactory results are obtained from using a steel band, as this latter does not catch on all the projecting twigs. Steel bands must be handled carefully to prevent their breaking; but the actual chainage obtained is too accurate for the purpose, and time must not be wasted in reading too precisely over the points. All chains or steel tapes suffer very much in tropical climates from rusting, and great care must be taken to have them thoroughly cleaned and oiled each evening after finishing the day's work.

The Levels.—Levels are taken along the traverse line with the ordinary spirit-level, the reduced level of each turning point and of intermediate chain points being recorded, also bench marks being established about every half-mile. If the levelman does his own checking back he can only cover about three-quarters of the distance traversed by the compass. On this account where speed is necessary an additional levelman is provided, called the check leveller, whose duty is to check the levels on the traverse and also do any cross-section levelling where the ordinary type of surveyor's level can be used for the purpose with advantage.

The Cross-Sections.—Where the ordinary level is found unsuitable for taking levels along cross-sections on account of the rapid changes in level and the denseness of the forest, necessitating the cutting out of a narrow passage-way, only 6 or 8 feet in height, the aneroid is generally found most suitable. The readings taken at points where chainages were marked on blazed branches of trees, being taken again at the same points when returning to the base line. This affords a useful and essential check on the first aneroid readings.[1] At

[1] The reason a level can only see very short distances in these steep, narrow passages, is that the natural roof or canopy of branches being about 6 or 8 feet

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times when the cross-sections are short a hand-level may be used with advantage. It is very important to select the position of the main traverse carefully when running through dense forest in order that the cross-sections may be as short as possible, as the "cutting" for these cross-sections is slow and tedious, generally taking about two days to cut a narrow pathway a mile long. But even when it is made at this speed, the cut branches may still prove a serious obstacle to taking measurements unless one or two extra natives have been employed to remove them as the chopping party advances.

The direction of a cross-section is generally taken by prismatic compass, though an ordinary pocket compass has at times been used for the purpose and given sufficient accuracy, where the cross-section line was not very long.

Astronomical Observations to Check Bearings of Traverse Lines.—It is important to remember on the main traverse to make astronomical observations for the true bearings of its lines every few miles. The differences of these true bearings should agree fairly closely with the differences between the corresponding magnetic bearings taken when traversing, any error being distributed equally among the lines intervening between those where the astronomical observations were taken. Thus, without regard for the lengths of the lines, each is given an equal correction. Although the bearings of long lines are more liable to error, the amount of difference is very small and may be neglected in these cases. The operation of finding the azimuth of any line by observations, or by the sun, is given in Appendix II. If the survey party is camped on the traverse line, or nearly, the azimuth may be obtained by finding the meridian by observations on circumpolar stars in the evening after returning to camp.

Plotting the Traverse.—The standard method of plotting traverses for railway preliminary surveys is by latitudes and departures. But as the stations on the traverses in tropical forests occur very frequently, it is considered hardly advisable to devote so much time and labour in getting the results of the field work plotted in the office. A method of plotting, which is much more rapid and at the same time quite accurate, is that of laying off the calculated courses from an assumed meridian. This is naturally a very suitable method of plotting lines whose directions have been taken by compass. Another method, not so accurate, but frequently used for this work, as its degree of accuracy is sufficient for the purpose, is the setting off of bearings by the help of a large protractor and parallel rollers. Although there is no check on the accuracy of the plotting by this method it

above ground changes its level with the ground surface, and cuts off short the range of view in foresight almost as quickly as the surface of the ground does in backsight.

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fulfils the requirement of having the field notes, taken during the day, plotted rapidly the same night to enable any deviations from the direction being followed to be made the following morning, if found necessary. Also the traverse can be plotted again within a few days on tracing paper and placed over the first traverse to check it, and correct any error which may have occurred during the first plotting. Having plotted the traverse the topography, both natural and artificial, are next put on the plan, and from the system of level points on cross-sections rough contours are interpolated. Thus a contour plan of the whole strip of country through the centre of which the traverse lay is prepared.

Office and Field Location.—On this plan the location of the railway is performed in the usual way (see Chapter V.). When, after some trial lines have been attempted on this plan in pencil, one is found which seems to be the best available, it is inked in and a number of offsets taken to it from the main traverse. These are carefully measured on the plan and "set out" by measurements from the traverse on the ground. Ranging poles are set at these points, and a passage through the forest between them is made, rather wider than that used for the ordinary traverse. In this passage way it may be found that poles which are supposed to be in line are not so, and require altering here and there a few feet, or possibly more, to bring them into line. In order, the more easily to give the direction for cutting the passage way from pole to pole, the bearing of the line is calculated from the plan. Then the instrument is set over one of the offset points, and the line of sight turned in the direction of this calculated bearing. This enables the chopping of branches to be carried forward without delay. If it is found that this direction does not strike a good average of the poles, it may be necessary to widen the passage on one side for the purpose of rectifying it.

It frequently occurs that the first line located in this way is found when tested by levelling to give a rather different profile or longitudinal section than that obtained from the topographical plan. This is due to the fact that it has not been possible to locate a large number of surface levels owing to the difficulty of obtaining them through the dense growth. Consequently a great deal of interpolation is necessary in order to sketch in the contour lines at all, and it is not unreasonable to suppose that even a skilled topographer will fail to locate them very near their true positions.

First Located Line as Traverse for Improved Topographical Plan.—On account of the above-mentioned difficulties the first located line on the ground is frequently used as a base from which to survey for a better one. This second location will probably be very near the first, but in order to find it a more minute topographical survey of a

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very narrow strip of country is carried out with hand-level and tape, levels at every chain length along the first located line having been taken with a surveyor's level. The actual amount of topography needed in this survey can be determined in the office when a complete set of levels along the first located line is available for comparison with those on the contour plan. Thus little more than the exact data required need be taken in the field, and the survey plotted rapidly, and a second location completed in a very short time and put in on the ground.

Final Location.—This located line ought to agree very closely in point of surface levels, with those obtained from the profile prepared from the topographical plan. If this is found to be so, and the line in every other respect seems satisfactory, it is permanently staked out, and the ground cleared for a width of 10 or 20 feet. This is to prevent the line being lost to sight through the rapid growth of the forest before construction begins. Later on it will need clearing for a width of 100 feet or more.

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CHAPTER IV

THE PRELIMINARY SURVEY

WHAT is generally understood by the term "preliminary survey," is the field and office work necessary for the production of a topographical map or plan of that strip of country which the reconnaissance pointed out as that within which the railway can be located satisfactorily. It is also understood that any additional strips of country which this more detailed survey points out as worthy of consideration should be included in the above-mentioned plan.

But, in a wider sense, the preliminary survey includes all operations which must be performed, after and with the help of, the reconnaissance survey, so as to reduce the field of selection for the final location from a number of possible routes to that of one route, included within a narrow strip of country not more than a few hundred feet wide; and further to provide a topographical plan of this strip so that the final location may first be projected upon it in the office, if necessary, before establishing it on the ground.

Determining the most Suitable Route.—It frequently happens that the reconnaissance has only decided upon two or three probable routes at certain places between governing points, and has left to the more accurate instrumental work of the preliminary survey the duty of selecting the most suitable. In these cases it is not necessary to survey each of these routes completely. There are certain difficult sections of each route which, when surveyed, will probably determine the most suitable one, generally that which gives the easiest ruling gradient. The least suitable route can usually be eliminated early, and if two routes remain of almost equal value the selection between them will be made on the results obtained from surveys of only the more difficult parts of each route.

In making this selection a number of points will be considered, such as the ruling gradient, the maximum curvature, the total length of line, the amount of earthwork and bridging and the nature of foundations available.

Where two routes are of almost equal value, it is necessary to prepare

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complete contour maps of the portions of each route surveyed in order the more carefully to weigh up their relative merits in the office.

The work of determining the position of the contours and other topographical data necessary for these maps, as well as for the complete preliminary plan, is best accomplished by running a traverse as near the centre of the strip as the natural features of the country permit. This traverse is run with great care, as the true position of every object is determined by measurements from it.

Trial Location Surveys.—Some engineers include under the term "preliminary survey" the running and recording of traverses over level and easy country, where very little deviation from the straight is necessary and where little or no contour work is carried out. These traverses which partake more of the nature of a first location, and from which primary estimates are prepared, are run with the view to having as little alteration afterwards as possible. It is in this respect that they partake more of the nature of those location surveys in which the principal work of locating is done in the field instead of in the office.

Objections to Trial Location Surveys.—It is only in the case of level and otherwise easy country such surveys can be economically carried out, and if there is any risk of increasing the cost of construction a topographical plan must be prepared. But as such stretches of easy country are rarely met with for any considerable distance except on prairie tablelands or sandy deserts, the custom of placing the preliminary traverse so carefully with a view to its final adoption as a first location involves too much waste of time in the field, except in such cases. The position of the first location is best and quickest determined by placing it upon the topographical plan prepared in the office from the data obtained in the preliminary survey. Therefore the topographical plan prepared during the preliminary survey must contain sufficient contour lines roughly in their correct positions to enable the best line to be projected on the plan, and also be sufficiently accurate that when this line is located on the ground it may be found to possess very nearly the properties of the projected line.

It may be as well to distinguish still more at this stage between the so-called preliminary location, above mentioned, where no topographical plan is prepared to indicate the best position for the line, the work being entirely done in the field; and the preliminary survey in which the topographical plan is prepared. For the former it is usual to follow as nearly as possible the general direction between limiting or controlling points, making long lengths of straight and the deflection angles between the tangents or straights as small as the surface will permit. In order to accomplish this it is frequently necessary to run into the side slopes of a valley at bends, or cut right through small hills or other obstacles.

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The running of such lines without prolonged delays needs a very capable transitman. He must be able to see at a glance the advantage of preserving his straight through such obstacles, in view of the subsequent features of the country about to be encountered. In rounding sharp bends it is advisable not to make the turn in one sharp angle, but to distribute it between two or three short lengths of straight.

This may seem at first a very straightforward and economical method of laying out a line. But in the field the conditions are not always as pleasing as the above short description might imply. Even in fairly open country large obstacles frequently obscure the view of the instrumentman, and if he laboriously attempts to pass any one of these on parallel lines, restoring the line to its true position after passing the obstruction, he may find that the line should have been deflected at a point some distance back. On such occasions, which constantly occur, the whole party is brought to a standstill, all stakes marking that part of the line are pulled out, and another attempt is made to start forward from a point further back. With what success this second attempt is likely to be met is also only a matter of surmise, but eventually a line is staked out which the engineer thinks is the best he can get, and it probably is having in view the difficulties he has had to overcome. But there is little justification for supposing that this or any line attempted on the ground is the best line which the natural features of the country can afford, because the line selected was that which seemed best in conjunction with the part already completed. On account of this there is little likelihood of that already staked out being altered in determining the position of the new portion, as in that case that which preceded it also might need altering. In this way the location of a road in the field is influenced too much by that portion of road already staked out.

Advantages of Locating on a Topographical Plan.—On the other hand, when locating a road on a topographical plan in the office, the lines which are first sketched on are of little moment when the subsequent ones are being fixed and are readily erased if found to connect badly with these, especially those lines which are tied within narrow limits in rougher or more difficult parts of the surface. The ease with which a large number of trial lines can be sketched on the topographical plan and the less suitable erased in favour of the more suitable is in itself sufficient reason for adopting this method in most cases. But, in addition, the opportunity of getting a broad grasp of the topographical features in a few minutes when studying the plan prevents the possibility of any trial line being omitted which may prove itself suitable in the end.

Occasions when the Survey for the Topographical Plan is the only Survey possible.—But to consider "location" in the field

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further : it frequently happens that the line must be run through a thickly wooded country. Here the distance a sight can be taken with an instrument is only a few yards without chopping down trees and undergrowth, and no general grasp of the slope and surface of the ground can be obtained without clearing the forest. But it is not possible to clear even a sufficient number of trial passages through the forest radiating from each turning point to enable the best line to be run in on the ground in this way. On the other hand, any line through a forest may suffice for the picking up of the topography on cross-sections and thus enable a topographical map to be prepared in the ordinary way.

The term "Preliminary Survey."—Thus the term "preliminary survey" generally implies the preparation of a topographical plan and the accumulation of other data such as: the nature of the rivers, the kind of forests and any geological information. But it must not be supposed that this precludes the need of employing a transitman skilled in railway survey work. There are many difficult questions to be decided in order to ensure that the strip of country through which the transitman lays his traverse is the best one and also that the traverse may occupy as near as possible the centre of it. This latter is advisable to enable the topographical data to be quickly and easily reached from the traverse line. The strip of country surveyed is usually about 500 or 600 feet wide, except in the case of steep side-hill work or in very rough or mountainous country, when it may be much narrower, as the surface level changes rapidly by small movements to the right or left. It may also be necessary in very flat or slightly undulating country to survey a strip very much wider than 600 feet, because any desire to change the elevation of the trial line on location, say by 5 or 10 feet, may need shifting to the right or left very much more than 300 feet if the surface has a very gradual slope. At the turning points of the traverse it is advisable to widen the half-width of the strip on the inner side of the angle, as the curve on the location will probably occur on that side. Fig. 18 (p. 82) shows a short length of topographical plan prepared during a preliminary survey. In the event of a very much wider area than one of 600 feet width being required, it is generally advisable to run a second traverse. From this the topographical data of half the area can be acquired, the other half being obtained by measurement from the original traverse. The advantage lies in the fact that running long cross-section lines from the traverse is liable to cause an accumulation of error both in the levels and in the direction, the data obtained near the traverse being much more accurate than that more remote. Consequently, on long cross-section lines greater care has to be exercised to obtain reliable data, which means the expenditure of a great deal more time on topography. This additional expenditure of time is the more

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undesirable, as the topographer at ordinary times takes longer to acquire his notes than the other members of the survey party.

Gradient

Generally, the first question which confronts the transitman or chief assistant when starting the preliminary survey is, What is the class of gradients which is likely to predominate on the railway? and, a still more important point, What is the ruling gradient?

Ruling Gradients.—The "ruling" or maximum gradient is the steepest gradient adopted on the road which is operated by the traffic conducted in the ordinary way.

Pusher Gradients.—When the gradients are steeper than the "ruling" gradient and therefore the ordinary trains cannot be hauled up them without additional help, they are termed "pusher gradients." This name is probably due to the custom of placing the assistant locomotive at the back of the train. But on "pusher" gradients the traffic is not always dealt with in this way. Frequently the train is divided into sections, two or more, which are hauled up separately and then reformed into one train. "Pusher" grades are only met with in mountainous country, where tunnelling or other heavy structural work is prohibitive on account of the cost.

Fixing the Ruling Gradient.—But the question of gradient, if not approximately settled on reconnaissance, must be dealt with in the early stages of the preliminary survey. Much depends upon the nature of the country as to whether it is possible to estimate during the reconnaissance survey, an approximate "ruling" gradient which will afterwards be found to agree nearly with what is obtained on the preliminary. If the country is fairly easy, it is not usually necessary to run a trial survey to determine the easiest ruling gradient possible, since the ordinary preliminary traverse follows a line of easy gradients and good alignment. As a rule the most hilly or rough country through which the line has to pass determines the ruling gradient for the whole system. And there is little advantage in adopting much better gradients in the easier part of the country, if much steeper gradients have to be adopted on other parts of the road; that is provided their adoption means increased cost of construction. Where any doubt exists as to the "ruling" gradient which can be adopted, the engineer-in-charge should point out at an early stage of the survey any portion of the strip of country requiring surveying, which is likely to affect the gradients on the whole line. And a rapid traverse should be run through this area and levels taken, but little or no topography. A number of parties may be engaged upon this work

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to hasten its completion, the work of each party co-ordinating with that of the parties nearest to it. It is not generally necessary to start these parties at the beginning of the survey. It is preferable, if possible, to survey a length of the ordinary traverse first, and remove the parties whilst the topographer is still in the field completing his work on the portion run by the transit party. In this way some time may be saved. But much depends upon the nature of the survey and how far the area needing to be rapidly surveyed is from the camp. If the distance is too far to carry the parties there and back each day, it may be found more convenient to start by encamping in the special district referred to.

The actual work on these rapid surveys which is being performed by several parties, should be under the direction of one engineer, preferably either the chief of party or his assistant. He should keep a constant supervision over the work of each party so that their ideas may be in unison as to the results obtainable. In this way the work can be performed more quickly and with little repetition. For, if it becomes evident that one or two parties cannot obtain an easier gradient than a ·8 per cent., or 1·0 per cent. (1 in 125 and 1 in 100) it is useless for the other parties to strive for a ·5 per cent. gradient.

Methods of approaching the Summit of a Valley by a Line within the Ruling Gradient.—In running the ordinary preliminary line the levelman should follow as close on the heels of the transitman as possible to enable the latter to change the direction of his line if he finds the gradients are not working out satisfactorily. Usually the line is run forward from one end, but if at any point it is intended to follow along the side of a valley up to its highest point or summit, it may be necessary to work back from this summit, to keep within the "ruling" gradient, and to carry this traverse along until it makes a satisfactory junction with the original traverse. Perhaps a more satisfactory method is to make a rapid survey with level and tape from a point on the original traverse to the summit at the upper end of the valley. On this traverse the bearings at turning points are best taken with a prismatic compass. In this way the difference in elevation between the point on the original traverse, and the lowest point at the summit through which the line can be run is obtained rapidly, and also the approximate distance. If from these results is found the need for a gradient steeper than the ruling gradient, it will be necessary to push the latter part of the original traverse gradually, and as the gradient permits, higher up on the side slope of the valley in order to keep well within the ruling gradient to the summit of the valley. It is necessary to keep well within the ruling gradient, because the chainage on the traverse, especially if it winds about much, will give a considerably greater distance from the entrance to the top of the

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valley than the more direct located line, and thus give a false impression of what gradient is available. It may frequently be thought unnecessary to discontinue the ordinary traverse in approaching a summit from a distance because of the valley having a very gradual slope. This fact, however, is very misleading, as the slope becomes much steeper at the head of the valley, and at the same time the valley becomes very small and leaves little latitude for raising or lowering the traverse on its side-slope. The chief disadvantage in working back from the summit is that the chainage runs the reverse way, and requires to be re-run in order to carry it forward continuously along the main traverse. It also requires the levels to be reduced to agree with those of the main traverse, because in starting from the summit it is necessary to fix a bench mark and assume a value for its elevation.

It has at times been found convenient to set the vertical angle corresponding to the ruling gradient on the transit, and to sight up the valley from a point on the main traverse to determine how high up on the side of the valley the main traverse must start in order that it may reach the summit within the ruling gradient. If in sighting up the valley it appears to be possible to find such a point, it is almost certain to give even better results than expected, as the traverse line when set out will follow a longer path than that of the line of sight of the instrument.

Alignment

The preliminary traverse is essentially a base line of the nature of a skeleton framework to which all topographical detail is attached. Its object is to enable this detail to be collected and recorded on a plan to scale. This being the case, it is necessary to run the traverse in the best possible position for obtaining this detail, provided in doing so no serious delay is caused in pushing forward the survey work. The traverse should follow the line of least resistance, which is near to the centre of the strip of country needing recording. There should be no hesitation in turning the line to the right or the left, if in so doing serious obstructions to the line of sight are avoided. If possible in avoiding these obstructions do so by turning a very small angle to the right or left, and return to the original line and direction by turning back another such small angle, as this has the effect of making the increase in length of the chainage due to the divergence a very small quantity.

If it is possible to make the traverse follow a very direct course—long lengths of straight and small deflection angles—so much the better, for then the length of the traverse will almost agree with that of the located line, and also gradients calculated from the traverse with

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those actually obtainable on location. In this case it is also possible to make a fairly close estimate of the cost of the road from the profile of the traverse itself. But any attempts to make the preliminary traverse coincide with the final located line in anything but the easiest country are misplaced efforts, because little is gained by such endeavours, when a topographical plan is being prepared, whilst on the contrary much time is wasted. It is not on the preliminary survey that the question of alignment should be dealt with, but on location ; though any unnecessary deviation from good alignment in running the traverse is to be deplored. But so long as the correct topographical data is collected the effect of such unnecessary deviation will have no influence upon the located line.

The Traverse.—This is a system of straight lines (as shown in Fig. 9) connected to each other in chain-link fashion, and measured

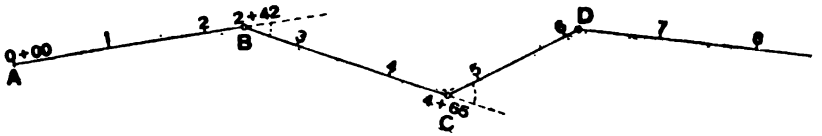

FIG. 9.

continuously from one end. Each chain length is marked on the ground and recorded, and the chainage of the ends of the links, or "turning points" as they are called, are both marked on the ground and recorded in the field notes. All turning points, including the starting point, and points used for the production of straights, where the straight is too long for a single sight, are fixed on the ground by driving in large pegs 2 inches or more in diameter. The chainages of these pegs are marked as also the chainages of the "station" or chain points upon labels of wood 7 or 8 inches long, driven 2 or 3 inches into the ground 2 inches to the right of the station points, and sloping back from the line so as to point to a point about 4 inches beneath the surface of the ground, vertically under the station point. The marking on the label should be on the upper side as fixed in the ground, and written from the pointed end upwards, so that any one walking along the traverse line from the beginning, may read the chainage of any point without having to turn back or turn the wood label over. The ordinary station points have no other marks in the ground. Fig. 10 shows a turning point indicated by ⊙, and its chainage 271 chains and 39 links.[1] If a level is taken at a station point the staff is put upon the ground 2 inches to the left of the label peg. In recording chainages

[1] If it is necessary to fix an intermediate instrument point in a long length of straight the label is marked with △ instead of ⊙.

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in field notes also no special mark is assigned to the ordinary station points, but in the case of turning points it is usual to place a circle to the left of the chainage, whilst a triangle is used for the similar purpose in the case of a point used for producing the straight. If it is impossible to fix a label peg at any point owing to the nature of the surface, the peg is usually fixed at the nearest point on the right or left of the line, where it can be conveniently driven into the ground. In this case the number of feet to the right or left of the line is marked on the label, as, for instance, 4′ 0″ R, or 6′ 0″ L, indicating 4 feet right or 6 feet left in addition to the ordinary chainage. Although the peg, if driven properly with its marking on the side of the centre line, will indicate whether the centre line is to the right or the left, it is as well to have a distinct understanding what is meant by R or L. Sometimes 4′ R is taken to mean that the centre line is 4 feet to the right of the peg, whilst at other times it is understood that the peg is 4 feet to the right of the centre line. This latter one seems the most rational convention and the one which should be adopted. These points which are marked on the ground to the right or left of the centre line should be carefully recorded in the field notes to facilitate the work of the levelman and topographer who follow after the chainmen.

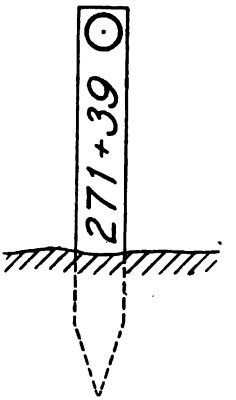

FIG. 10.

Angle Measurements at Turning Points.—The angles at each turning point is carefully measured by the transitman, the actual angle taken being that between the production of the previous straight and the new straight, and indicated in the notes by a deflection Right or Left, according to whether it is as at B or C in Fig. 9. These angles must be measured very accurately, for an error in any one of them will swing out of its true position all the subsequent part of the traverse when plotted by a radial motion round the point of error, each subsequent turning point being out of its true position by an amount proportional to its distance from the point where the error occurred.

In order to ensure careful reading of each angle the "*repetition*" method should be adopted. The operation is as follows :—

Set the vernier to read zero and releasing the lower or limb clamp, turn the telescope to sight accurately on the back hub, making a fine adjustment with the lower tangent screw. Now transit the telescope and unclamping the vernier plate, direct the telescope to sight on the new hub forward. Here clamp the vernier and make a fine adjustment on the new hub with the upper (or vernier plate) tangent screw. Now

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carefully read the angle and record it. Next, with the plates thus clamped, transit the telescope and, releasing the lower clamp, bring it to sight again on the back hub (this time with the vernier not reading zero, but the angle just recorded). Now transit the telescope and unclamp the vernier plate and direct the telescope again to the new hub. After clamping the vernier plate and making a fine adjustment with the tangent screw, the deflection angle is read and this time it should give an angle double that previously read if both readings were taken with equal care. This angle when halved should be a more accurate reading of the actual deflection angle, and, at any rate, have a value within 20 seconds of that first obtained. For very careful work a third repetition is sometimes taken, but this is not advisable with the ordinary field theodolite and should be unnecessary on a preliminary traverse. Having thus read the deflection angle and determined its nearest value, record beside it in the field notes the letter R or L according to whether the deflection was to the right or left.

Again, before leaving the station read the bearing of the compass needle and enter it in the field notes, at the same time observing whether the difference in bearing of the two lines agrees to within about half a degree of the recorded deflection angle. This is a further check upon the reading of the deflection angle. If the latter does not agree within 30 or 40 minutes with the bearing, the angle should be observed again.

Producing a Straight Line by the Transit Theodolite.—In producing a straight line, which is often necessary on a traverse, it is necessary to set zero on the vernier and sight upon the back hub. Then transit the telescope and give the line to the leading or head chainman. The latter drives a hub accurately and centres it on the line and the transitman, unclamping the lower clamp, turns the telescope round to sight again upon the back hub, but without transiting the telescope. Then, having fixed the lower clamp and made a fine adjustment on the hub with the tangent screw, transit the telescope and sight forward again. If this line of sight does not pass through the mark previously put upon the hub the chainman fixes a new one, and finding the point midway between the two marks drives a small nail into it. If the instrument is in good adjustment the two points found thus will almost coincide. This method, which should always be adopted in producing a straight line with a theodolite, is termed "*double facing*" or "*double centring*," the object being to change the Ys or horizontal axis of the telescope from left to right for the second observation, so that if the axis and line of sight of the telescope are not accurately at 90° to each other the error resulting therefrom may take effect on both sides and thus enable the true line to be found midway between them.

Laying down Meridians.—It is advisable to lay down a meridian

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at the starting point of the traverse so that the true bearing of each straight line of the traverse may be obtained. This should be repeated at the end of each 20 miles run or so, in order to check the results and correct for the change in the azimuth. The meridian may be satisfactorily laid down by the circumpolar star method, or by observations on the sun (see Appendix II.). In this way the bearings of all lines become known, and as their lengths are also known, the positions of their extremities (the turning points) with respect to the starting point, are fixed by known co-ordinates with respect to axes through the starting point. These are generally referred to as "latitudes and departures."

Running a Second Traverse.—The need for running a second preliminary line may occur when traversing a valley, the sides of which alternate in their suitability for the located line. Thus it is often advisable to run a traverse along both sides of a valley at various points and perhaps for a considerable length. But if the river is wide and thus needs expensive bridge work for crossing it, very little crossing and recrossing is likely to be attempted, and unless the properties of the surface most suitable for railway construction seem to become transferred from one side to the other probably no crossing at all will be attempted. Another instance where a second preliminary line nearly parallel to the main traverse may be run with advantage is where a strip of country wider than 600 or 700 feet needs to be surveyed. If the country is rugged and seems to be cut up a good deal and possibly covered for the most part with forest or underbrush, it is so difficult to draw any conclusions from a narrow strip that a double or treble strip must be surveyed. It must not be supposed that rough country is unsuitable for railway contruction, for the ridges and valleys follow so close on each other that a little care in fixing a gradient line will generally enable the cuttings and embankments to balance each other within the limits of free haul.

It is usual to name each traverse line by a different letter of the alphabet. For instance, if the "A" line is the main traverse, the first additional traverse will be called the "B" line, and each subsequent one by a different letter. The chainages where these auxiliary traverses depart from and return to the main traverse are especially marked for each line and are termed "chainage adjustment points" or "equalization points." For example, if "B" line starts away from "A" line at chainage 127 + 34 and returns to it at chainage point 158 + 19, the chainage on the "B" line at this point being 31 + 40, these chainages are shown in the notes as follows :—

"B" line 0 + 00 = 127 + 34 "A" line
„ 31 + 40 = 158 + 19 „

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It is also usual when referencing these to mark in the notes of "A" line in red or blue pencil—

"A" line 127 + 34 = 0 + 00 "B" line
" 158 + 19 = 31 + 40 "

Plotting the Traverse

When the general direction of the traverse is known it is possible to decide what office meridian to assume. The principal difficulty is to keep the traverse when plotted from running off the edge of the roll of paper. As long as the railway is taking a pretty direct course, there should be little difficulty in keeping long lengths of it upon the same roll.

The Latitude and Departure Method.—It is first necessary to find the true bearing of the general direction. This can be done very approximately, as a meridian is laid down at the beginning of the traverse. The assumed or office meridian is taken at 90° from the general direction which is now known. From this meridian the latitude and departure (or longitude) of each turning point is calculated and a table of latitudes and departures prepared. Such a table is shown in Table V. (p. 84). The operation of plotting these points on square-ruled paper is very rapid. On the edge of the roll at starting, opposite the beginning of the centre line, zero is marked and each 500 and 1000 foot measured above and below this point is also marked. These latter are usually put on in pencil every few feet along the plan. Also each 1000 foot is marked from the zero along the centre line. Thus in plotting each point the longitude and latitude of some point on the paper is always near-by and enables the position of the turning point to be found at once. Fig. 18 (p. 82) shows the traverse given in the latitude and departure Table V., and which was plotted in this way. This method of plotting by latitudes and departures is the most accurate known, and has the advantage that if the plan gets distorted by damp or any other cause, the true position of each point is unaffected, as it always preserves its correct relative position to the sides of the square enclosing it. However, the preparation of the latitude and departure table entails more time and labour than many engineers care to afford.

The Courses set off from a Meridian Method.—Another method which is strongly advocated is that of setting off the course of each line from a meridian. This may be done by setting off by the tangent-offset method[1] a number of the calculated courses from the

[1] The tangent-offset method of plotting an angle is that of extracting from a book of Tables the value of the tangent of the angle to two or three places of decimals, and setting out, from the point where the angle is to be deflected, along

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meridian to start with, and transferring each to its correct place on the plan at the end of the preceding line by means of a straight-edge and set-square, or parallel rollers. A still better method is to have the assumed meridian as a centre line down the middle of the roll, and the true direction of each calculated course with respect to this line plotted in pencil at the beginning of the roll. Then each of these angles can be set in turn on the clinograph and transferred to the position of its line on the plan. This is a simple and rapid method of plotting with a considerable degree of accuracy which is maintained throughout, as there is no accumulation of error.

Plotting Methods to be Avoided.—Preliminary traverses should never be plotted by means of the protractor, as the angle so obtained is not sufficiently accurate; nor by setting off the direction of each line by a tangent-offset from the preceding line, as any error throws out of position all the subsequent part of the traverse, magnifying the error at any point in proportion to its distance from the erroneous angular point. As the traverse is plotted each station point is marked by a little tick on the side of the line, and each fifth one numbered, also the chainage of the turning points is marked on (see Fig. 9, p. 44).

The Levels

Over the traverse line, on the ground, as marked out by the chainmen, the levelman takes his readings, recording the level of every station point and turning point, and taking intermediate readings where sudden changes occur in the surface of the ground. A bench mark must be established at the beginning of the survey, either from those on the survey of the preceding portion, or by means of barometric observations, or even by giving it an assumed value. From this the level of each station point is obtained, and other bench marks are established not more than a mile apart.

The Profile.—A longitudinal section, or profile, of the traverse is prepared each day as the work proceeds to the same linear scale as the traverse, generally 400 feet to 1 inch, and a vertical scale about twenty times as coarse, frequently 20 feet to 1 inch. This profile provides the means of seeing approximately what gradients can be attempted, and if it is advisable to alter the position of the traverse, or run another traverse branching off from the main traverse, either on the higher or lower side of it. On account of this valuable property, it is

the production of the previous straight 100 or 1000 divisions to some scale, and erecting a perpendicular to the line at the end of these divisions, and setting off on this line the corresponding number of divisions as given by the tables for the particular angle. The point thus found, when joined to the starting point, forms the required angle.

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usual to plot the results of the day's levelling at night, and have the plotting completed before the chief of party surveys the general progress of the work in the office.

The Topography

Now that the levels of each station point are known for a certain distance the topography over the length can be obtained. The topographer usually makes a note the previous evening of the reduced levels of the station points from the levelman's notes for a length sufficiently long to provide a good day's work (see chainages and levels on the edge of the page of Topography Field Notes, p. 53). With this data at his disposal, and a good hand-level, he is able to work on lines perpendicular to the traverse line, and on both sides of it, and obtain approximate levels of the natural surface of the ground at any points he may require (p. 53 only gives the notes for the left side of the line). Some topographers work entirely alone without any help. They measure their distances by pacing, and find points on the 5-foot contours by sighting points on the ground with a hand-level, and pacing up to them. A careful topographer can do good work in this manner and very quickly, but there are many cases where it is impossible to see the surface of the ground, as in thick undergrowth, and in such cases much guesswork has to be indulged in. An error of a foot in sighting any particular spot on the ground would not be a matter of serious moment. But where such a point is used as an observation point to sight the next contour, and the same conditions exist there, after a few such repetitions the degree of accuracy might fall below the standard even necessary for this kind of work.

If the country is neither open, nor the surface fairly smooth, it is advisable that the topographer have a rodman, provided with a rod and tape, to help him. This is also as well on steep slopes, as there pacing is very unsatisfactory, the tendency being to take larger paces going down than uphill. Also in going downhill from the centre line, it is much easier to find the contour points with a staff than to pace forward a certain distance, and attempt to find the right point from which to sight back on the spot which has been vacated. Taping is certainly tedious and should be avoided, except on steep slopes and where pacing is difficult to perform. Any method of quickly obtaining data is of great help to a topographer, as it enables him the more fully and accurately to sketch in his note-book those topographical features of which his observation has afforded him a mental grasp, before the operation of levelling and measuring has dimmed his memory of them. It is very important that the cross-section lines upon which the topographer works should be approximately at right angles to the traverse line, as

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otherwise the topography at their extremities may be seriously misplaced when they come to be plotted on the plan, especially if the ground slopes in a diagonal direction. A little care in the use of an optical square will usually cure this.

Topographical Notes and Methods of Plotting them.—As the object of the preliminary survey is to obtain a topographical plan of a strip of country, any form of topographical notes which can be accurately and quickly transferred to the traverse plan is preferable to one which, though accurate, may involve much work in the office. For instance, a system of spot levels, whose positions are carefully shown by being dotted in on co-ordinate or square-ruled paper, are sometimes transferred to the traverse plan, and the contours sketched in by the draughtsman, interpolating them as he thinks best.

These are generally transferred to the plan by sketching on the latter in pencil cross-section lines and parallel lines at equal distances, as shown in Fig. 11, the squares so formed representing the same area

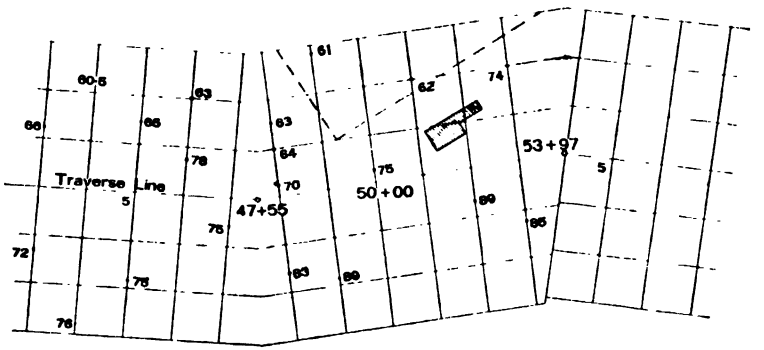

FIG. 11.

as the squares in the topographer's note-book. [*Note.*—This method, as shown in Fig. 11, can be used also with advantage where the topographer has already sketched in the contours on his notes in the field, as the latter can then be sketched in rapidly by the draughtsman, the squares guiding the direction of his pencil.]

When doing this sort of work, care must be taken to see that the sides of the squares in the field note-book represent the linear measurement to the same scale as is being used to plot the traverse in the office.

Probably the most satisfactory, and certainly the most expeditious method of dealing with the topography, both in the field and office, is to make the field notes in the field in the form in which they are to appear on the plan (see p. 53), and transfer them directly to the plan from the field book, or after tracing them on thin paper off the field

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notes, and then transferring these to the plan, either by the use of carbon paper, or better still by blackening the back of the tracing paper with a large soft black pencil or crayon. The soft black pencil leaves only a faint outline on the plan, but quite sufficient to enable the draughtsman to line it all in completely and afterwards erase easily all the black marks due to transferring through the tracing paper. The best method of all is to have a special field note-book, composed of a large number of small semi-transparent sheets of square-ruled paper. If any difficulty is experienced in getting this thin paper ruled in squares, a stiff sheet ruled in the same squares, but more heavily, may be used under a plain sheet clamped in the book as here described. It is necessary that the plain sheets have small round holes punched out of the middle points of the sides. Into these holes four small steel bayonets, fixed in the stiff cover, fit, and the whole is clamped in position at the top and bottom of the note-book. But here it is necessary to rule a line on the plain sheet to represent the traverse line, and mark the station points on it. This may either be done in the office or in the field. The topographical notes sketched on this semi-transparent paper are used directly by the draughtsman in the office for transferring the notes to the preliminary plan. Each sheet from the field notes is blackened on the back of the leaf by a soft crayon, and then placed in position on the plan, the draughtsman going over the topography with a stilo or hard pencil. The field note-sheet is then returned to the pocket inside the top cover of the note-book. All these sheets should be numbered by the topographer before making the field notes. It is better to have the transparent square-ruled sheets loose instead of being bound into a book, as the right-hand page is used for topography on the right of the line, and the left-hand page for that on the left. It is thus more convenient to keep these two together on one sheet, having one number for both. A space should be left in the middle of the sheet in which the chainage and reduced levels are entered.

This method of preparing a topographical plan is very accurate and expeditious, and removes much of the objection raised to such plans on the ground of time and expense.

The Duties of the Members of the Survey Party

The *engineer-in-charge* and his *assistant*, if there is one (this is preferable in order to keep the party going at full swing and cause no delays), will mark out for the transitman and chainmen the direction they are to follow. This is best done by the assistant, but he must receive his instructions from the chief as the results of his investigations of the country ahead of the survey party. Thus it is

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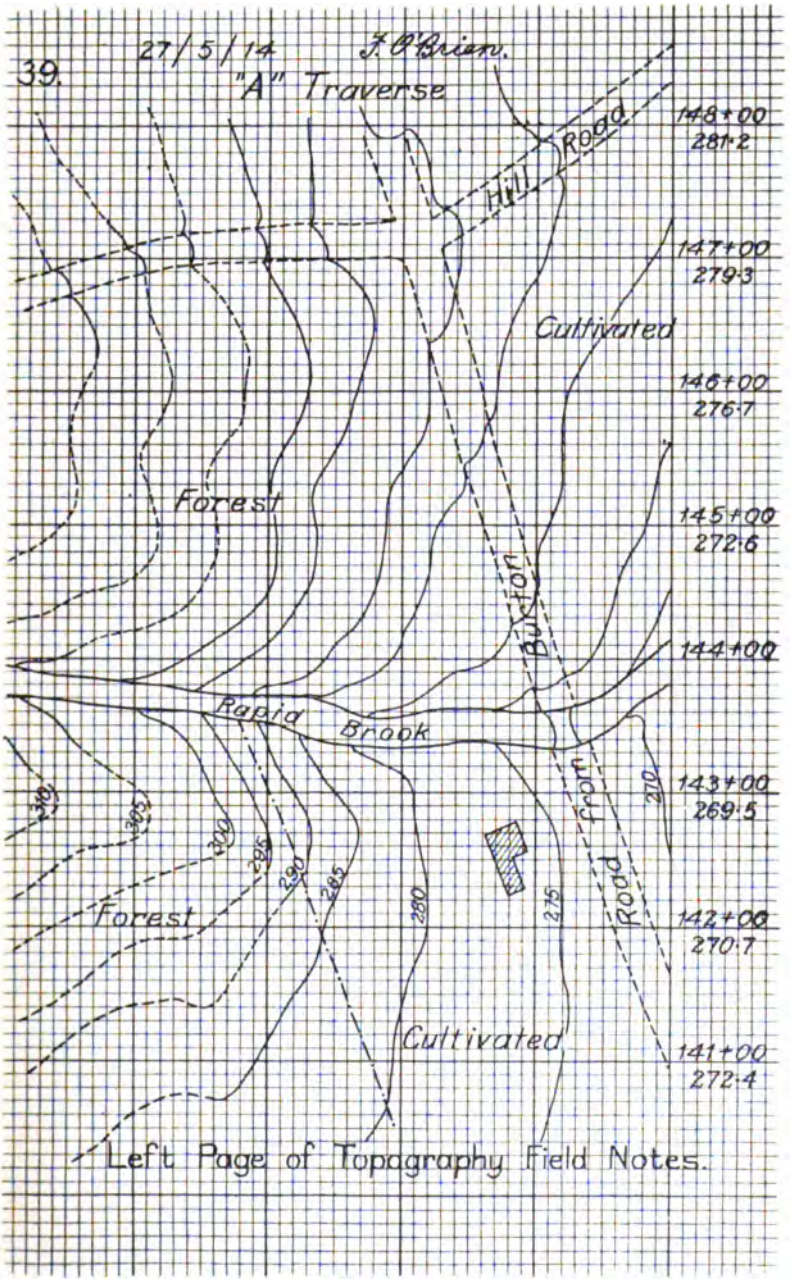

Fig. 12.

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necessary that the engineer-in-charge should keep his work well in advance of the party so that the assistant may have instructions for each day's work the night before, or when the party takes the field in the morning. The work of the chief on these investigations, and the kind of information supplied to the assistant, have been dealt with in Chapter II., p. 23, Second Reconnaissance (for details).

The Assistant Engineer.—He should be either an expert topographer or else very familiar with topographic work. He should also have had plenty of experience upon railway surveys. These attributes are necessary so that he may be able to make the best use of the information supplied to him by the chief. His principal duty should be directing the transit party. For this purpose he will keep ahead of the chainmen and axemen, and will mark out the direction to be taken by the transitman. This work will probably keep him fully occupied, for a transit party having its direction marked out for it will push ahead very rapidly except where timber is thick. Also the assistant will need to examine the terrain for a width of at least 300 or 400 feet on each side of his line. This means that he must pick up his points quickly from the data supplied by his chief, and push on without much delay.

He should be responsible for the work of each member of the party. On this account it is advisable when a new party is starting on a preliminary survey not to attempt to go rapidly for the first week. This will enable the assistant to supervise carefully the work of each member of the party, and, if necessary, drill any members of it into the correct routine and methods of procedure. Until he has confidence in the work performed by each member, he will always have the uncomfortable feeling that inaccuracies may creep in anywhere, and the part of the work which can be relied upon is uncertain. In this way it is better to waste the first week than to push on for some time rapidly from the first, and then find all the work has to be re-done.

The information to be supplied to the assistant engineer is best conveyed to him at night by the chief in the office. This should be done after the plan and profile of the day's work had been plotted. Thus, with this information and the topographical sketches made by the chief engineer, the course to be followed by the traverse can be discussed, and any alterations in the line already run or additional side lines suggested.

The Theodolite or Transitman.—Much of the work of the transitman has been given already under the heading "The Traverse," but a few details of his work must be here mentioned.

In a general way his duties are to read all angles and record them in the transit book as well as any other details of alignment such as rivers, roads, or other lines crossed, the bearings of the directions of

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which will need noting. He also assists the draughtsman in the office in the evening to plot the traverse.

Upon the transitman depends the speed of the party. He must work rapidly and be accurate, and in order to do satisfactory work he must be able to adjust his instrument and keep it in good condition. As the strain of constantly sighting through the telescope with one eye is great, it is as well for him to accustom himself to use both eyes. This will be appreciated by any one working an instrument in cold windy weather. The strain on one eye causes it to blur too much for taking reliable readings.

The routine followed by the transitman is approximately as follows: The leading chainman having fixed and centred a hub or turning point, the transitman starts forward with his instrument to occupy it. Whilst he is thus getting established for measuring the next deflection angle, the chainmen start forward chaining from the new hub in a direction pointed out by the assistant engineer. Two or three of the subsequent stations in this line may be lined in by the instrument, but it is unnecessary to line in every point. This method gives the transitman very little latitude in varying the direction of his line, but it saves time, especially on long lines, by enabling the chainmen to get ahead. The transitman picks up the direction by sighting on a pole, picket, or some other sight put in by the assistant for the axemen and chainmen to line themselves in upon. If this is not visible he will find it necessary to sight upon a few of the station points put in by the chainmen, and make average line with them.

As the transitman has to read the deflection angle accurately by repetition, he will only stop to give line to the chainmen when looking forward.

A code of signals must be adopted for instrument work, as with the telescope long sights can be taken, and it would not be possible to give instructions by shouting. Thus it is common to indicate that the object sighted upon should be moved to the right by holding the right hand up above the level of the head, and waving it gradually to the right until the object has been moved sufficiently. If the object needs moving a great deal, the result will be attained as quickly by more violently waving the right hand by a jerking movement to the right. The left hand is moved similarly to the left when the object is to be moved in that direction. When the object is in its correct position both hands are raised above the head. If the leading chainman holds a ranging pole horizontally over his head while he faces the instrument, it is generally understood by the transitman to mean that he is ready to fix a new turning point, which needs more careful work on the part of the transitman in giving the line for driving the peg, and afterwards for centring it with a copper nail.

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The form of field notes taken by the transitman are shown on pp. 58 and 59.

The Rear Picketman.—Associated with the transitman is the *Rear Picketman* or *Flagman*, whose duty it is to hold a ranging pole on the previous hub for the transitman to sight back on. He is sometimes required to keep a record of the number of hubs upon which he has given a backsight.

The Chainmen.—Their duties must be considered independently.

(1) *The Head Chainman* is the most important, and with the transitman is responsible for the speed of the party. As his work is at the head of the line, he is usually responsible for the work of the axemen and stakemarker. [The stakemarker is generally an axeman, who when not engaged upon marking the chainage upon labels of wood, or driving them or the hubs, gives a hand with the other axemen in clearing the line. He should be able to make clear figures in a bold type which can be easily read.]

The head chainman is responsible for the accuracy of the chaining. His method and that of the rear chainman must be the very best, always with a view to the greatest accuracy, but also the least waste of time. The greatest source of error is generally expected from the chaining. About the best results which can be expected from a chain party is no greater error than 1 foot in a thousand. As this degree of accuracy is much inferior to what an ordinary field transit can produce, the accuracy of the results much depend upon the chainmen. They should each carry a plumb-line for use whenever needed. The head chainman lines himself in with the previous station and the transit instrument. He should make sure to get himself nearly on line before calling on the transitman for line. He should also avoid constantly asking the transitman for line to direct the axemen. It is sufficient to fix at the end of the line a ranging pole with which and the instrument the axemen can line themselves in. If the latter are cutting through thick undergrowth a second ranging pole should be fixed on the line near them so as to give them the line more readily, as the transit may be at a considerable distance, and out of focus with near objects of foliage. When chaining railway curves, the head chainman should become accustomed to set himself approximately at each station point before calling for line from the instrument. This he will easily do by pulling the chain in the line of the two previous full-chain stations, and setting off with his ranging pole the offset, the amount of which he will have measured previously when the chord joining the first clear chain length of curve could be used for finding the chord offset. Having set off the approximate position of the new station

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he will swing the tape or chain into line with it, and wait for the instrument to fix the point accurately in position.

It is the duty of the chainmen to test and adjust their chain or tape every few days with the standard of length kept in the office.

It is at times necessary for the head chainman to select the best position for a hub, when the assistant engineer is not near. For this purpose it is well for him to know something about the working of the instrument, so that the sight forward and also back from the new hub may be sufficiently clear.

(2) *The Rear Chainman* must watch carefully the chain or tape when being pulled forward by the head chainman to free it when caught in any obstruction and also to straighten any bent links. He must hold his handle firmly against the pin or arrow marking the station point when the head chainman is pulling the chain into good line. At the same time he must avoid concealing the view of the other end of the chain from the instrument. This is the more difficult on straights, but on curves can be easily rectified by keeping on the outside of the curve. If starting the chain from the hub at which the transit is set, he must be careful not to touch the legs of the tripod or allow the chain to do so. This is difficult when the chain is being strained from the other end. The rear chainman checks the marking on the labels at each station when he arrives at it, and then calls out to the head chainman the chainage there. The head chainman checks this with the label prepared for the next station. This double check is very important, as the tendency to error in stake-marking is considerable. The rear chainman has the personal care of the chain or tape and the arrows, and must clean them each evening upon returning to camp.

When a hub occurs at the fractional part of a chain length, commonly called a plus, the whole chain is pulled forward, and the distance measured with that part of the beginning of it, the latter part of the chain being used for fixing the next station point on the new direction.

The Levelman.—He must be able to take his levels quickly and accurately, and also be able to make all the necessary adjustments of his level which are possible in the field.

He must take the level of every station point and turning point on the preliminary traverse, and at any intermediate points where the surface slope changes. At all river crossings or depressions where bridges or viaducts will be needed special care must be taken in levelling along this line crossing the valley; spot levels must be taken more frequently and their positions accurately taped. Such accuracy is needed so that profiles of these crossings may be plotted to a much larger scale than that of the general profile. The levels of high-water

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July 14th, 1914.
J. Forbes.

LINE "A"

Stations.	Vernier readings. Vernier A.	Vernier readings. Vernier B.	Deflection angles.	Compass readings.	Calculated courses.
△ 107 + 00					
⊙ 95 + 67·3	360° 00′ 00″	180° 00′ 00″	12° 47′ 15″ R.	N. 41° 00′ E.	N. 41° 1′ 35″ E.
	12° 47′ 00″	192° 47′ 30″			
	25° 34′ 30″	205° 34′ 30″			
	38° 21′ 30″	38° 22′ 00″			
	12° 47′ 10″	12° 47′ 20″			
△ 88 + 00					
△ 79 + 00					
⊙ 69 + 37	360° 00′ 00″	180° 00′ 00″	13° 7′ 40″ L.	N. 28° 00′ E.	N. 28° 14′ 20″ E.
	13° 7′ 30″	193° 8′ 00″			
	26° 15′ 00″	206° 15′ 30″			
	39° 22′ 30″	39° 23′ 30″			
	13° 7′ 30″	13° 7′ 50″			
△ 58 + 00					
⊙ 47 + 14·4	360° 00′ 00″	180° 00′ 00″	23° 16′ 25″ R.	N. 41° 30′ E.	N. 41° 22′ 00″ E.
	23° 16′ 30″	203° 16′ 00″			
	46° 33′ 30″	226° 32′ 30″			
	69° 50′ 00″	69° 48′ 30″			
	23° 16′ 40″	23° 16′ 10″			

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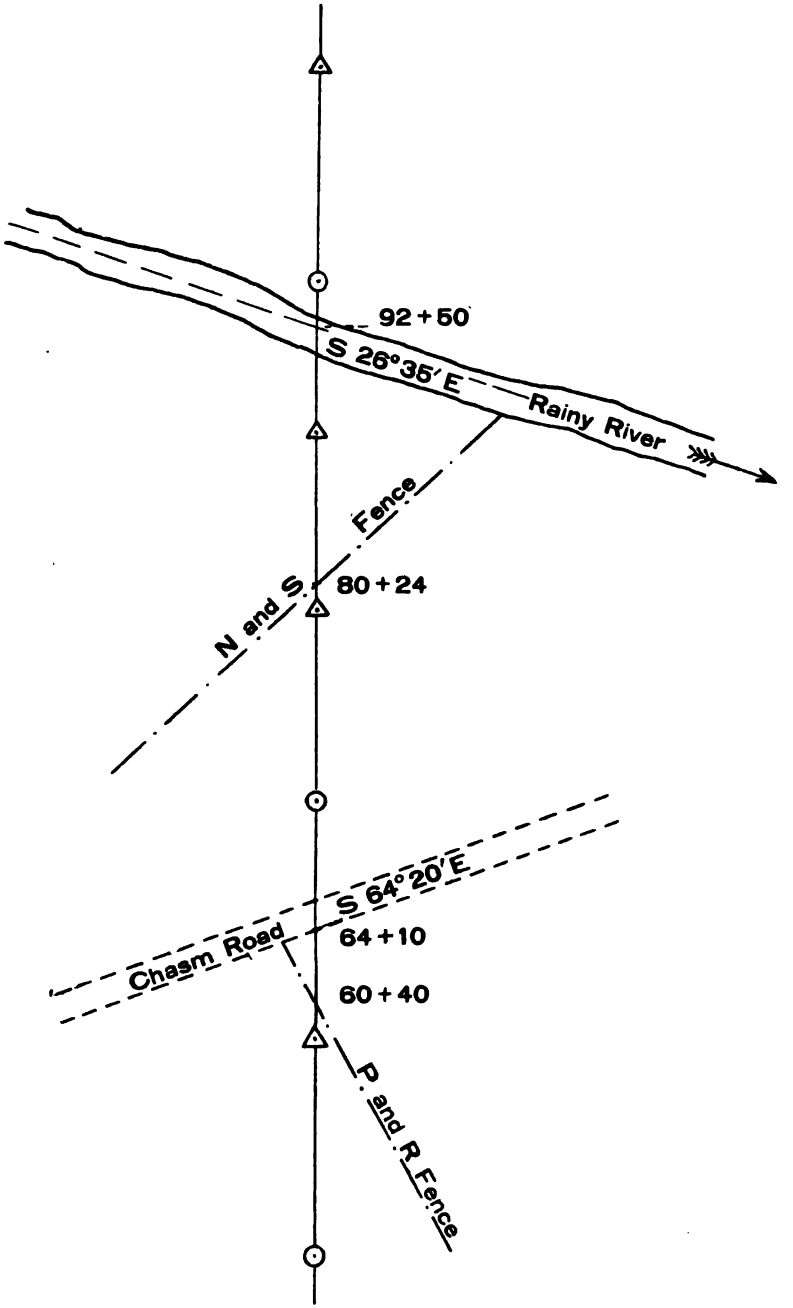

FIG. 13.—Field notes.

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marks, as indicated by deposits along the banks, must also be taken. He must note any changes in the material to be found in excavation which may show itself on the surface, and must record all rock outcrops, and any such information as will supplement that which the assistant or the topographer will have collected. He must establish bench-marks at least every mile, and in order to save any risk of delay in keeping up to the pace of the transitman it is advisable for him to take not more than half a mile of levels before checking back. In this way if any mistake has occurred, there is only a short line of levels needing to be re-run in order to locate it.

If the levelman has an intelligent rodman it is well for him to use a staff with a target attached; this enables the readings taken on all changing points, hubs and bench-marks to be doubly checked. This is effected by the levelman first taking a reading without the target and recording it on some scribbling paper, and then making the rodman fix the target and take a reading of it and call out the result. If these two results agree to the second decimal place, he enters the reading in the level book and proceeds. All bench-marks must be recorded to the one-thousandth of a foot. A record of these specially-taken readings and the reduced levels of the points on which they are taken should also be recorded by the rodman. A full and clear description of all bench-marks must be given in the level notes under remarks, and reference made to any book or map from which the value of the reduced level was obtained. See Level Notes, p. 61.

It is usual for the levelman at night to plot a profile of the portion of traverse levelled by him during the day. In doing this he has the assistance of the rodman, who calls out the levels and their chainages whilst he marks their positions on profile paper.

The horizontal scale of the profile will be the same as that of the topographical plan, but the vertical scale is usually twenty times as coarse. Thus where a horizontal scale of 400 feet to the inch is used on the plan, the vertical scale on the profile will probably be 20 feet to the inch.

It is a wise precaution when taking a careful reading on the rod to make the rodman swing the rod to and fro past the vertical position in the vertical plane passing through the instrument. The smallest reading obtainable during this motion of the rod is the same as would have been obtained had the rod been held steady and truly vertical.

For form of Level Notes see p. 61.

The Rodman.—The rodman has much to do with the speed of the levelman. On this account he should be energetic and have discretion in selecting changing points for the levelman. This judgment in the slope of the ground is a valuable asset in a rodman, as it enables the levelman to take the most out of the length of his rod, and reduce the number of "settings up."

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TRAVERSE LINE "B"

Back sight.	Intermediate or rod.	Fore sight.	Collimation or H.I.	Reduced level.	Distance.	Remarks and sketches.
5·55			251·705	246·155		B.M. on base of oak tree on left 52 feet from station 1 + 70·5, and 45 feet from station 1 + 00.
	8·47			243·23	1 + 00	
	7·33			244·37	1 + 70·5	Turning point.
	8·46			243·24	1 + 70·5	At centre of road.
	8·23			243·47	2 + 00	
	5·23			246·47	3 + 00	
	6·23			245·47	3 + 00	At centre of road.
	3·27			248·43	4 + 00	
1·43		·76	252·375	250·945		To shift.
	1·39			250·98	4 + 56	Turning point.
	2·01			250·36	4 + 56	At centre of road.
	2·92			249·45	5 + 00	
	13·71			238·66	6 + 00	
0·73		15·63	237·475	236·745		To shift.
	7·08			230·39	7 + 00	
	12·67			224·80	7 + 85	Turning point.
5·31		12·78	230·005	224·695		To shift.
	5·96			224·04	8 + 00	
	3·61			226·39	9 + 00	
2·16		3·96	228·205	226·045	10 + 00	To shift. On road near station.
	4·54			223·66	11 + 00	
	6·50	6·50		221·70	12 + 00	
15·18		39·63		246·15		
		15·18		221·70		
		24·45		24·45		

FORM OF LEVEL NOTES.

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As stated previously under "Levelman," he must read and record the target readings taken at all turning points, and keep records of the reduced levels of these points and all bench-marks. He will not record the levels of ordinary station points or intermediate surface points, but he should be able to pace the distance to each of these intermediate points from the nearest station point, as it is unnecessary, where the ordinary plotting scale is used, that their positions should be nearer the true ones than 2 or 3 feet.

It is his duty to fix all bench-marks either by driving a peg solidly into the ground for the purpose, or better still if a tree is available to blaze a small portion of its trunk near the ground where a low part projects beyond the rest, and to cut a rectangular wedge out of the top of it so as to make a horizontal surface on which to rest the staff. Many drive a nail so as to lie flat on this surface, and help to identify the bench-mark after some months or years. Where the tree is young and no projecting part on the trunk exists, a small blaze on its side and a large nail driven in so as to project about 1 inch, will serve the purpose well. Peg or stake bench-marks are not advisable in cold climates, as the frost will tend to draw them out of the ground. The position of a bench-mark is sometimes defined by its distances from two points in the traverse line.

The Topographer.—All the details of the topographical plan, the positions of which are determined chiefly by their distances from the traverse line measured along cross-section lines at right angles to it, must be determined by the topographer. These details ought to be only what are required or what are likely to be required for the purpose of the location. If more is collected than is required, it only leads to the delay of the whole party. Topography is generally a source of delay to the preliminary survey party, a first-rate topographer covering with difficulty little more than two-thirds of the distance covered by the transit party in the day in most terrain. On this account, when the whole survey party has advanced the line as far as is convenient from the camping place, it is usual to make other members of the party take topography to help the topographer to finish up to the end of the traverse line.

The topographer must have sufficient knowledge of railway survey work and also of topography, to recognize through which part of the area under his observation the railway will most likely pass, and pay careful attention to his notes of that part, whereas his notes of the remainder may be sketched in rapidly and with much less degree of accuracy. The authors have found the most satisfactory, expeditious, and economical arrangement in getting topographical data was afforded by the use of a hand-level or hand-level and clinometer (such as an Abney level), and having one man as rod and tapeman carrying a 15-foot

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rod and a metallic fibre linen tape. (The rod should be telescopic or else be hinged in the middle and fold up so that the graduated faces of the two parts will be in contact and thus protect each other.)

Some topographers prefer to work alone with a hand-level by sighting on points on the ground, and moving up to each of them before taking the next sight. This may be convenient on the high side of a cross-section, but very inconvenient on the low. A rodman, when he has got accustomed to pacing distances, and to the method of working which is being adopted, will not be any source of delay, and will save a great deal of doubtful calculating and enable much more accurate results to be recorded. He will keep carefully on the cross-section line when set out at right angles to the main traverse line. The authors, when working with a rodman, have found it convenient to make the collimation or level of the eye occur at an even contour. Thus if the station point had a reduced level of 207·4, they would move about till they could read on the staff placed on this station point 2·6 for a collimation of 210·0 or 7·6 for 215·0 or even 12·6 if they were working on the high side. This might seem a waste of time to many, but can be done very rapidly where contours are required fairly accurately. How frequently cross-lines of levels of this kind should be taken depends upon the nature of the terrain. If the surface is changing rapidly, the cross-sections should be run every 100 feet, and the contours sketched in for the part between as they appear to the topographer. On the other hand, in flat or gently undulating country, lines of levels may not be required oftener than every 400 or 500 feet. A typical page of field notes is given in Fig. 12, p. 53. [Note the chainages and reduced levels at these chain points entered at the edge of the leaf from the levelman's book the previous night. These are actually in the centre of the two pages.] Much of the topographer's work and methods have already been set out on pp. 51 and 52.

The Draughtsman.—The operation of plotting the preliminary traverse has been pretty fully described already; so the arrangement of office plans and the other duties of the draughtsman will be found in the next chapter dealing with "Location." Although an office location is made every few days during the preliminary survey, partly as a guide to the probable position of the final location and also to ensure that the topographical plan covers the area required, the methods adopted in this location are the same as for the general location on long strips of country. Accordingly, it is better to deal with the whole subject under that heading.

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CHAPTER V

OFFICE LOCATION

LOCATION may be conveniently dealt with under two headings, viz. Office Location and Field Location.

Office location, which is the immediate guide to the position of the line when set out on the ground, and which also provides the permanent records, is carried out on plans and sections or profiles of various scales according to the purposes for which they are required.

Scale of the Topographical Plan.—The working topographical plan, upon which the line is projected, is frequently to a scale of 400 feet to an inch, though the authors have found 200 feet to an inch more satisfactory, and in mountainous or rough country 100 feet to an inch.

The Profile or Longitudinal Section.—This has the same horizontal scale as the topographical plan, and a vertical scale about 20 times as coarse. Thus the 400 feet to an inch plan will probably have a vertical scale on its profile of 20 feet to the inch, and that for the 200 feet to an inch plan of 10 feet to the inch.

None of these scales are fixed by hard and fast rules, each railway adopting those it prefers, the particular nature of the country and the work fixing them more or less. A simple decimal scale for a plan has generally been found advisable to facilitate its production in the office, and its use in the field. Thus, 100, 200, 400 and sometimes 800 feet to the inch have been adopted very satisfactorily, and in the field a 2-foot rule can be used on any of these plans conveniently. If possible, one scale should be adopted for the whole length of topographical plan, and certainly not more than two, as the locating engineer, when projecting location on a plan always to the same scale, becomes possessed of what may be called "absolute" distance, and knows at a glance the effect of any small movements he may make with the locating line.

The Inch to the Mile Plan.—In order the more readily to get a general grasp of the country, and to see long stretches of the line on one plan, an inch to the mile plan (or sometimes 4000 feet to an inch)

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is always prepared. The quality of this plan depends upon the existing maps available of the country, as the data subsequently added to such a plan can only have been obtained in a rough-and-ready way by the engineer when engaged upon reconnaissance, and supplemented later by the more accurate data of the narrow strip of preliminary survey. Eventually the located line also is plotted on this plan, and a profile prepared. These are also useful for directors of railway companies and other officials to whom reports of the work generally may need to be submitted.

Sections or Profiles of Preliminary and Location Lines.—It is usual to prepare a profile of the traverse lines of the preliminary surveys, as well as of each projected location, to the scales already described. On each of these sections trial gradients are projected, the final one showing the bridge openings, positions of borings to determine the nature of materials in cuttings and also foundations, and the amounts of material in cuttings and embankments.

The Topographical Plan.—The plotting of this plan by latitudes and departures, by laying off calculated courses from an assumed meridian, or by other methods, is fairly fully discussed in Chapter IV. With regard to the paper to be used, it is very advisable that this plan should be plotted on a roll of square-lined paper on account of the importance of being able to examine a long strip of it without delay at any moment, and also of preserving the plan in good condition. Some engineers prefer separate sheets on account of the convenience of taking them into the field for the purpose of entering the topography directly on them. Any advantages this may bring with it which are very doubtful, especially in wet weather, become almost infinitesimal if the method of collecting and plotting topography suggested in Chapter IV. is adopted. Again, the difficulty of inspecting a long strip of the topography on account of having first to piece the separate sheets together, and fix them down on a long table, becomes a very serious drawback, and actually hinders as free an inspection of the plan as is desirable for careful location.

Contour lines should be inked in in black on the plan in almost every case, each fifth one being a line of double the thickness of its fellows. Even in the case of artificial topography being present, which needs lining in in black, there is little confusion caused by this practice on account of the artificial topography being more uniform. However, where much of this is present, it is advisable to pick out the contours in a different colour. Probably the colour which bears most rubbing or erasing work to be performed on a plan without disappearing is burnt sienna, which also shows up well if a tracing of the plan is used for blue printing or other photo-printing operations.

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The Draughtsman.—In the field office there is generally plenty of scope for a quick and accurate draughtsman, and one capable of doing a good uniform type of lettering. He must be able to calculate the quantities in cuttings and embankments, and prepare distribution curves and estimates.

It may be necessary for him, in addition to plotting the ordinary field notes each day, to prepare approximate estimates of two or three trial locations. This often needs a rapid and experienced draughtsman.

It is usual for the levelman to plot at night the profile of the portion of line he has levelled over during the day. Also, the transitman frequently assists the draughtsman in plotting the traverse. The topography notes can generally be left to be plotted next day, unless some portion of them is needed for the discussion of an alternative route to be run the next day. In such a case they can be very rapidly plotted at night after the traverse is completed.

Most of the operations performed by the draughtsman will be given in the subsequent pages on location. His methods of preparing the topographical plan have already been given in Chapter IV.

Projecting the Location upon the Topographical Plan.—The topographical plan, which is as complete as possible with all the data collected in the field, should have the level numbers of the contours entered on it in lines from the highest to the lowest at frequent intervals along the plan, so that the actual level of any contour line may be quickly observed when projecting the location.

A first-class railway line should be composed of long lengths of straight connected together by short flat curves (of small degree) on gentle gradients, so that in operating it the maximum of speed may be maintained with the minimum of work. Such lines can only be constructed if the natural features of the country permit. For instance, a line following along the base of a range of hills or mountains, may be obliged to skirt the hills so much in order to avoid too heavy gradients that instead of producing long lengths of straight connected by short flat curves, it will probably become a succession of long curves connected by short lengths of straight or tangents. In such cases the quality of the road depends upon the natural formation of the bases of the hills. If sharp curves require to be used in one place, they will most likely be needed in other places, and rarely even with heavy expenditure in construction can the curves be flattened sufficiently to improve the quality of the line very much.

Having in view these two types of lines, between which most others graduate, it is necessary that the locating engineer be equipped for locating either. Thus, if he first locates tangents or straights, a few yards of black glazed cotton or silk thread is indispensable. This when stretched over a length of the topography and fixed in position by paper

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weights, shows very clearly a possible position of a tangent, and enables a study of the contours it hugs and those it cuts, to be made. If, on the other hand, curves predominate, either a box of transparent celluloid railway curves[1] must be available, or a better arrangement is to have a number of the principal curves used on railway work inked in on strips of tracing cloth and used on the plan, since with these latter it is much easier to see where they cross the contour lines, and to study the lines on each side of them at the same time. These are used much in the same way as the thread line, being placed on the topographical plan, and shifted about until one is found which lies conveniently in position amongst the contour lines. If a straight-edge is placed tangentially with the previous curve, the new curve can be kept touching this line, and at the same time be moved along it so as to adjust itself to the topography. The length of tangent between the curves should be maintained as long as possible, and not less than about 500 feet between reverse curves. This length is not necessary between curves deflecting in the same direction. When fixing curves during office location it must be constantly kept in mind that the sharpest curve on the same or a steeper gradient as other curves determines the quality or operating power of the line. No attempt to convert a 4° curve into a 3° one by heavy constructional work should ever be attempted, on a line where other 4° curves exist on as steep a gradient in the same direction. But a great deal of work on location is worth while performing if it will enable a road with flatter curves to be produced. However, even curves must be subservient to gradients, and it may be found when projecting the gradients on the profiles that the easier curves produce much steeper gradients, thereby forcing the location back on to the previous location with sharper curves. The resistance to operation caused by curvatures and the consequent compensation on gradients is dealt with later in this chapter.

Before starting to stretch the black thread line on the topographical plan the locating engineer has a number of things to consider. He must observe the governing points through which the line is obliged to pass, and knowing the approximate difference in level between the immediate two of these points between which he is projecting the location, and also knowing the country intimately between these points, he will decide how the gradients should be distributed so as to gain this difference in moving from one point to the other. For instance, if the difference in level and distance between two governing points would require a uniform gradient throughout of 26 feet in the mile, or about ·5 per cent. in passing from one to the other, it will probably be found that for much of the distance between these governing points

[1] Table XXII. gives inch-curves to any given chains radius, for scales of 40, 20, 10 and 8 feet to inch. These refer to a 66-foot chain.

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the country will not support so steep a gradient, whilst for other parts a steeper grade is imperative.

Thus 15 per cent. of the distance may require a 1·0 per cent. gradient
30 „ „ „ „ ·8 „ „
37 „ „ „ „ ·3 „ „
and 18 „ „ „ may be level.

Keeping some such figures as these in his mind, which he will have roughly estimated when passing over the ground, and with the help of the profile of the preliminary traverse, he will endeavour to stretch his thread line so that it will cut the contour lines at the rate of gradient required. Again, he may mark out on the plan secondary governing points which these gradient limitations impose upon him, and strive by stretching lengths of his thread line to make the best-looking alignment possible to pass through the nearest of these points without cutting too many contour lines. His eye, when experienced at this class of work, will suggest very nearly what the profile will look like.

He must now proceed still farther with the alignment, and see what he can do in reaching the next tying point. This may prove a more difficult problem, and it may be found necessary to project this part independently, and revert to that already fixed for readjustment. All these projections are only effected with great care, numerous positions of the thread line being studied before any one is adopted, the slightest movement of the line often causing a great improvement in the location.

In addition to what he sees possible on the topographical plan, he is influenced by other data, which he has collected, and which, though having an important bearing on the location problem, has not been shown on the topographical plan.

Thus each portion of the projected location has an important bearing on that which came before it as well as that which follows it; and on this account frequent examinations of long lengths of the projected location must be made on the chance of improving it by altering any part of it. In this way the first operation in projecting a location is performed with infinite care by going backwards and forwards over each section of it until the best possible alignment is obtained approximately on the gradients required.

The Profile.—The next operation is to get the profile corresponding to this location plotted, and to project the gradient lines on it. This is done also with the black thread, the latter showing very clearly in each position the depths in cuttings and embankments. No time should be wasted in putting on an approximate formation line or gradients for the whole length of the profile. The cuttings on the profile should then be marked, with the nature of the material each

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contains, by the locating engineer and the draughtsman be left to calculate out the quantities in cuttings and embankments, and the distribution of materials, amount of overhaul and also of borrow and waste. When this information is ready it will at once be seen what improvements must be tried in the positions of the gradient lines. When this has been done it will be easily seen where the heavier construction is needed, or where the overhaul is likely to become a large item, or what other things are likely to lead to expense. When these have been reduced as much as possible on the profile any further remedy can only be effected by returning to the topographical plan. Here, generally, an improvement may be brought about at many points without seriously damaging the alignment. This is important, since the first line located should be the line of best alignment and easy gradients. Afterwards, if this is found too expensive, the line should be cheapened. Even if it is known that the line must not be an expensive one, it is well to always locate the best possible at first, as this may prove to be an inexpensive line, but even failing this it is the best guide to finding the most satisfactory line possible at the limited cost.

After thus revising the location a new profile is prepared and the gradient lines projected on it. If, when the quantities are taken out, it is found that the remedies sought after in some cases have been overdone, whilst other defects have not been sufficiently cured, and if further projections of the gradient line will not adjust these, the topographical plan must again be visited and adjustments made in the alignment location, which will remedy as much as possible these faults. In this way, by frequent projection on both the topographical plan and also the profiles, may a line be located which is as near as possible to the best which the natural features of the ground permits. These operations are typical of those performed frequently in the field office of the locating engineer. Figs. 18 and 19 (p. 82) show a satisfactory located line which was found very near to an unsatisfactory one.

Elementary Trial Locations.—It might be well for the young engineer when studying location projections to start by laying a system of lines of constant length (say 5 to 10 inches) on the topographical plan to a uniform gradient. Each of these lines gains the number of feet by cutting the contours, which the gradient requires for that length of line. In this way the plan will have a chain of short lines roughly on the gradient required. This gives the idea of the sort of line on a topographical plan which will produce such a profile. But this chain of short lines on the topographical plan will not give very good alignment. He must now lay his string line on the plan and find the best alignment which can be put on so as to hug the chain line as much as possible, cutting it frequently to enable the cuttings and embankments to balance as near as possible.

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There is yet another way of studying location with the same object in view. It is that of projecting a trial line on the topographical plan and preparing a profile and gradient. Then for each 100 foot or 200 foot point on this projected location take from the profile the grade level, and on the corresponding rectangular cross-section line mark the surface point, between the contours, having about the same level. If the system of these points be joined by a dotted line to distinguish it from other lines, it gives approximately the surface line in which the formation plane or sub-grade outcrops. It is evident that the nearer the projected location can be moved towards this line the less will be the amount of material in cuttings and embankments. Before projecting the dotted line it is advisable to project an easy gradient on the profile, no matter how heavy the construction resulting may be. In this way the dotted line marks out the direction towards which the located line may be moved with the best results. In Fig. 18 is shown such a dotted line.

It is intended by the suggestions given in these last few pages to give the young engineer some idea of the lines upon which he may work in projecting locations. But each individual location has many problems to be solved which are peculiar to itself, and to solve which require the exercise of judgment, but such cannot be dealt with here.

Office Operation of Plotting the Profile.—When the located line is drawn in in pencil on the topographical plan, the draughtsman prepares to plot the profile on the roll of profile paper. This latter is generally square-ruled, giving the same horizontal scale as the plan, but with the vertical divisions suitable to the exaggerated vertical scale adopted. Along the bottom edge of the roll the chainage at each 1000 feet should be marked on the vertical lines corresponding, and having selected as a datum a level about 50 or 100 feet below the lowest contour likely to be cut by the located line, mark each 50 and 100-foot contour on its horizontal line every couple of feet along the roll, so as to catch the eye quickly when plotting. The locating engineer will now take a strip of the topographical square-ruled paper (about 1 inch wide) and a few feet long. This strip should be cut from a roll of the same paper lengthwise, so as to be subject to the same expansion or contraction as the topographical plan due to moisture or other atmospheric conditions. It should also be tested on the topographical plan to see that it agrees with it. The strip should now be graduated every few inches showing 500 or 1000 feet intervals. Now, laying its edge along the located line, the engineer calls off the chainage of each point where a contour is cut, naming the contour level at the same time, to the draughtsman, who finds the point on the profile paper by the ordinary co-ordinate method, moving horizontally along to the correct chainage, and then vertically to the point of correct elevation in feet. Through the system of points so found

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the draughtsman draws in, by an easy motion of the hand, the surface line of the profile.

Fixing the Gradient.—Upon this profile the black thread line is stretched and weighted down in various positions to enable the effect to be studied. In this work the costly parts to be avoided are—(1) heavy cuttings or embankments; (2) cuttings and embankments not balancing; (3) heavy overhaul; and (4) costly bridges or other structures.

If there is no overhaul and the cuttings and embankments balance, the cost of forming the road bed will be practically the cost of excavating the cuttings; that is exclusive of culverts and bridgework.

So the engineer in projecting a grade line has to steer between having his gradient too steep and his constructional work too costly. Eventually, after several trials a grade line is marked on the profile. Then, guided by the data collected on the traverse line or by the topographer, the engineer marks on the probable nature of the material to be found in the various cuttings, the widths of openings for bridges, etc.

Volumes in Cuttings and Embankments.—The draughtsman now prepares a table of cross-sectional areas and volumes per 100 feet in cuttings and embankments, or for intermediate lengths where changes in the surface occur. In doing this he usually assumes that, at the depth of cutting or embankment as shown on the profile, the surface line is a level one.

This much facilitates the working out of the quantities. Table XXIII., Appendix II., gives the volumes per 100 feet for ordinary side slopes of 2 to 1, 1½ to 1, 1¼ to 1, 1 to 1, and ½ to 1 for various widths of road bed in common use, and for every foot of depth up to 50 feet, where the surface is assumed to be level. The error from making this latter assumption is generally compensating, and not more than 5 or 6 per cent. But where there is a continuous side slope in the surface in one direction, this level section method of calculating volumes for both cuttings and embankments is liable to considerable error, often 10 to 15 per cent., the error always causing the calculated amount to be on the small side. Thus a cross-section on a side slope may have an actual area of 200 square feet, but by the level section method of calculating the area only 170 square feet may be obtained.

Having found the areas of all cross-sections, the volume between each consecutive pair of these is calculated by multiplying the distance between them by the average of their areas, *i.e.* where the volume tables are not used. For the purpose of projecting location this level section method is quite accurate enough and is quickly performed. But when the final location is completed and set out on the ground the

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true forms of the surface at the cross-sections must be obtained either when putting in slope stakes or sometimes independently. Then the cross-sections are plotted and their areas accurately measured. The true volume must then be calculated more accurately by the prismoidal formula, or the areas of the cross-sections measured by a planimeter.

Earthwork Material—Swell and Shrinkage

All material when excavated changes its volume and in almost every case increases it. But after it has been tipped into an embankment and given time to consolidate it is found that all varieties shrink, and some, such as clay and light vegetable soil, even more than they have already expanded. As the problem to be dealt with is the amount of excavated material required to fill the embankment volume, it is necessary to estimate what the swell of the material from any cutting will be before attempting to balance the cuttings and embankments in the profile when locating upon it the gradient line. It is that volume, which a certain quantity of material will fill after it has become consolidated, which is of primary importance. Table III. gives the percentage change in the volume excavated when it becomes consolidated in an embankment.

TABLE III.

Material Shrinkage and Expansion Table.

The volume in excavation in each case is 1·00.

	Volume after excavation and before subsidence.	Volume after subsidence.
Rock, large pieces . .	1·50	1·40
Rock, small pieces . .	1·70	1·35
Chalk	1·80	1·40
Clay	1·20	·90
Light sandy soil. . .	·95	·89
Gravel	1·00	·92

This table is given more as a guide to what the probable swell or shrinkage may be of material experienced in practice. But so many grades and mixtures of material are met with in the earth's surface, that each particular one can only have its percentage swell or "swell constant" approximately estimated until some practical experiments prove more nearly what its true value is.

Quantities Distribution Table.—In Table IV., p. 74, all the material dealt with is clay, which was of a similar kind throughout, consequently the same "swell constant" is used all through the table,

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namely, 10 per cent., the sign being negative because it is shrinkage. Column No. 4 gives the cubic yards in the cutting volume or the material in place, and also in the embankment for each length between the chainages given in column 1 taken from a table for level sections. In this case the road was made for two tracks, and was taken 30 feet both in cutting (see Fig. 14) and embankment, the side slopes being $1\frac{1}{2}$ to 1 in both cases.

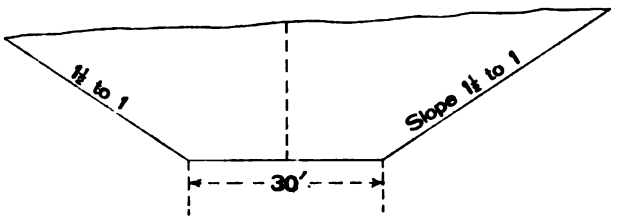

FIG. 14.

The quantities in column No. 4 are marked + for cutting, and − for embankment. Column No. 5 gives the "swell constant," and column No. 6 contains the quantities from column No. 4, corrected for the swell constant. Thus the + quantities in column No. 6 give the actual number of cubic yards of consolidated filling available for the embankments whose volumes are given with the minus sign. Column No. 7 gives totals of consolidated material available, or in deficit at each chain point. Thus the second item + 258 is the sum of + 52 and + 206. But the fourth one + 274, is the sum of the first three items in the previous column with the fourth one deducted from them. Thus column No. 7 indicates the actual amount of material available, or in deficit at any point provided no waste or borrow has been resorted to anywhere between the first chainage point and the point in question.

THE DISTRIBUTION CURVE OR MASS CURVE

If the quantities in column No. 7 are plotted as ordinates with their chainages for abscissæ on square-ruled paper, a curve is produced which is commonly called the "mass curve." The "mass curve" has many interesting and useful properties which assist in determining how the material in cuttings and embankments shall be distributed most economically. Fig. 15 shows the "mass curve" plotted from the data in column No. 7, and beneath it is shown the profile from which the data in the table on p. 73 is obtained. The same horizontal scale, for distances, is used for both diagrams, so that points on the mass curve are vertically over their corresponding points on the profile. The vertical scale of the mass curve is generally expressed in thousands of cubic yards per inch. The zero line from which these vertical ordinates

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TABLE IV.

Table of Quantities.

1. Chainage of station.	2. Depth of cutting in feet.	3. Depth of embankment in feet.	4. Cubic yards in cutting + in embankment −.	5. Kind of material and swell constant.	6. Cubic yards corrected by swell constant.	7. Total quantity of material available at chainage.
40 + 00	10·0		+ 58	clay	+ 52	+ 52 at 40 + 50
41 + 00	3·5		+ 229	− 10%	+ 206	+ 258
+ 30	·0	·0	+ 69	,,	+ 62	+ 320
+ 50		3·5	− 46		− 46	+ 274
42 + 00		5·8	− 320		− 320	− 46
+ 70	·0	·0	− 260		− 260	− 306
43 + 00	5·0		+ 104	clay	+ 94	− 212
44 + 00	7·0		+ 867	− 10%	+ 781	+ 569
45 + 50	7·5		+ 1683	,,	+ 1516	+ 2085
47 + 50	15·0		+ 4161	,,	+ 3745	+ 5830
49 + 00	17·5		+ 5137	,,	+ 4624	+ 10454
52 + 50	6·0		+ 8018	,,	+ 7217	+ 17661
+ 75	2·0		+ 133	,,	+ 120	+ 17781
54 + 00	·0	·0	+ 146	,,	+ 132	+ 17913
55 + 50		6·0	+ 479	,,	− 479	+ 17434
56 + 25		21·0	− 1745		− 1745	+ 15689
59 + 50		19·0	− 13789		− 13789	+ 1900
60 + 00		25·0	− 2566		− 2566	− 666
61 + 50		20·0	− 7361		− 7361	− 8027
62 + 00		35·0	− 3629		− 3629	− 11656
64 + 40		11·0	− 12648		− 12648	− 24304
+ 50	·0	·0	− 78	clay	− 78	− 24382
65 + 50	22·5		+ 5313	− 10%	+ 4782	− 19600
67 + 00	28·0		+ 10386	,,	+ 9348	− 10252
72 + 50	·0	·0	+ 9746	,,	+ 8774	− 1478
+ 75		19·0	− 389		− 389	− 1867
73 + 25	·0	·0	− 432	clay	− 432	− 2299
77 + 00	12·0		+ 4161	− 10%	+ 3746	+ 1447
81 + 00	·0		+ 3596	,,	+ 3094	+ 4541
84 + 00		11·0	− 2335		− 2335	+ 2206
87 + 00		11·0	− 5682		− 5682	− 3476
88 + 50		·0	− 1538		− 1538	− 5014

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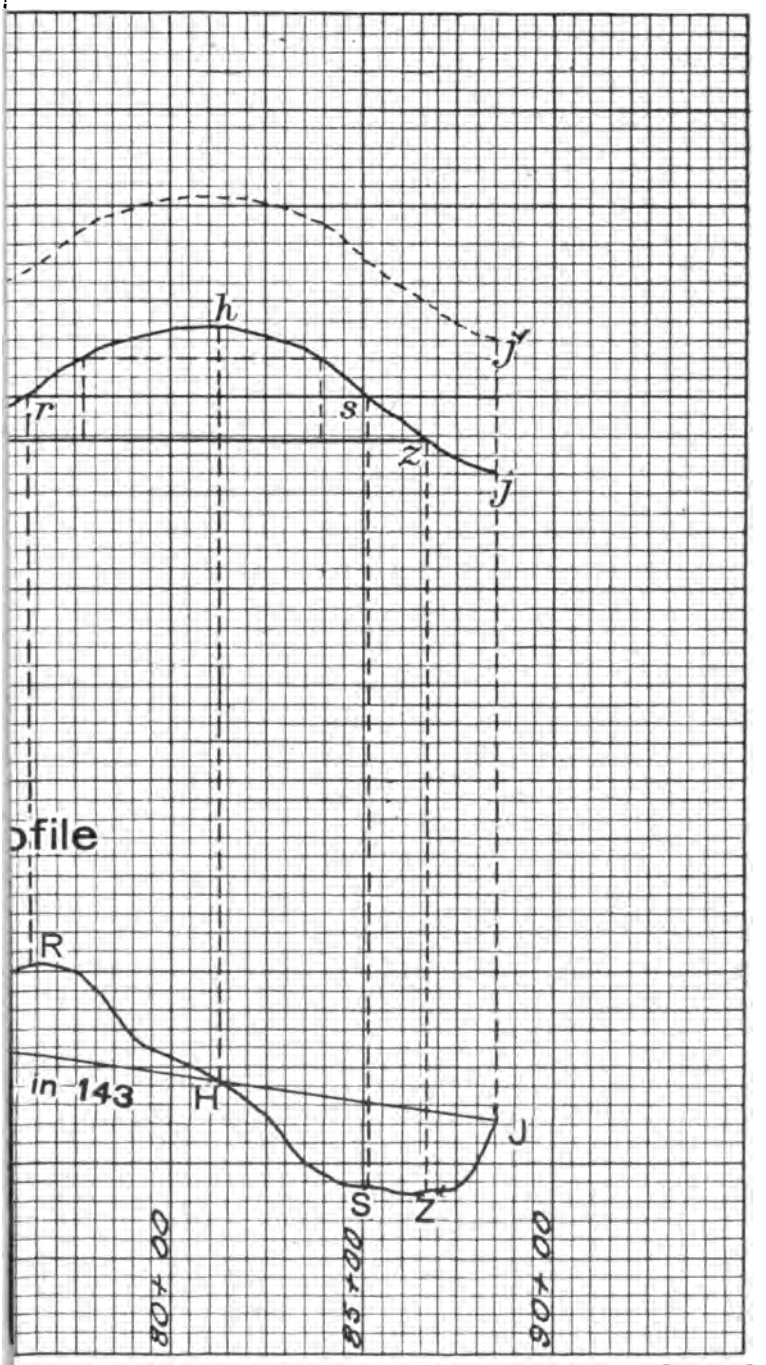

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are measured is called the "base line." Since AB in the profile is a cutting [it is part of a cutting, A being a point up to which all previous material was balanced, so that no material must be carried to the left of A. It is also assumed for this section of profile that no material is to be carried past J] the table starts with + quantities, and accordingly the mass curve begins by rising at *a*, and reaches its height at *b*, corresponding to B in the profile. That is all the material in the cutting AB has accumulated up to this point, but now the embankment BC begins to reduce that accumulation until at *o*, corresponding to O, it has been reduced to zero, thus indicating that cutting AB provides sufficient material to fill the embankment BC as far as O. After O the total volume becomes negative as is shown by the mass curve falling below the base line. The line continues to fall as far as *c*, corresponding to C, the end of the embankment. Then the curve begins to rise as fresh material is obtained from the cutting CD, until the curve again cuts the base line at *p*; thus indicating that the material obtained from the cutting between C and P is sufficient to complete the filling of the embankment BC. Now from P to D the material will accumulate, and the ordinate at *d* will represent the volume occupied by all the material from the cuttings between these points. As we pass beyond D the negative volumes of the portions of embankment DE will begin to reduce the accumulation of material indicated by the ordinate at *d*, consequently the curve line will drop until at *q* it once more crosses the base line, indicating that the accumulation has disappeared, or that the cutting beween P and D will only provide sufficient material to fill the embankment as far as Q. Thus it may be observed that the curve rises over the whole length of a cutting, and reaches its maximum height where it changes from cutting to embankment, and that it continues to fall throughout the length of an embankment, only turning to rise again where the embankment changes to cutting. The same rule applies whether the curve is plotted from right to left or from left to right. But it must only be used in the direction from which it has been plotted. That on Fig. 15 has been plotted from left to right. The rest of the embankment from Q to E will have to draw its material from the following cuttings EF and GH. This is indicated by the mass curve dropping below the base line. Thus it can be seen that as long as the curve is above the base line all material is being carried forward, but when the curve drops below the base line the material must be carried backward.

Balancing the Material.—The points in which the base line intersects the curve are balancing points of the material. Thus, between A and O, material is balanced as indicated by *ao* on the base line, also between O and P, and between P and Q, Q and R, and R and S.

But other horizontal lines cutting the curve also give balancing

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points of material. The horizontal line *xygz*, tangent to the curve at *g*, also gives such a system, *xy* indicates that the cutting from E to Y will just fill the embankment from E to X, also between Y and G all material is balanced and between G and Z. But the effect of such a balancing line when coupled with the first portion of the base line *aq*, is to leave a gap QX for which no material is provided. This would require to be borrowed. And if material could be conveniently borrowed near QX, this distribution arrangement is not open to any objection on that score, as it diminishes the total amount which must be borrowed between S and J due to the curve dropping below the base line at *j*, as the amount required to fill between S and Z is the same as that between Q and X, each being indicated by the same length of ordinate at *x* and *z*. In this way a number of horizontal lines may be tried to find the most economical balancing line. If the embankment between K and L is replaced by a bridge or viaduct, no material will be tipped between these points, and therefore the deficit of material at K will not be increased until L is passed. On this account the mass curve between *k* and *l'* is a horizontal line, and the remainder of the curve to the right of *l'* will be the original curve moved vertically upwards through a distance equal to *ll'*, equivalent to the amount of material saved by replacing the bank by a bridge. This curve is shown by the dotted line *l'j'*. It may be noted that this arrangement requires material to be wasted as shown by *j'* being above the base line instead of borrowed, as in the case of *j* below the base line. It should be noted that a bridge between K and L, unless constructed beforehand, or unless a temporary trestle bridge is constructed across the site, will prevent any material from being carried across the strip KL, thus preventing the filling of the embankment between K and Q with material taken from the cuttings EF and GH.

Distance of Haul by the Mass Curve.— If a horizontal line *mn* be drawn across a loop of the mass curve, and a parallel line infinitesimally near to it or at a distance, say equal to the ordinate for 1 cubic yard, be also drawn, then the strip of mass curve *mn* between these two lines indicates that one cubic yard at M is hauled through a horizontal distance *mn* and deposited at N. Therefore the area of the narrow strip *mn* represents the product of cubic yards by distance, or if the distance is expressed in chain lengths, say chain cubic yards or chain yards. If for each cubic yard represented by the ordinate at *d*, horizontal lines be drawn as at *mn*, a strip for each cubic yard of material will be produced showing the length of haul, which differs for each one; and the sum of these strips or total area *pmdnq* will represent the total cubic yards multiplied by the average distance or haul. If this area be next divided by the cubic yards in the ordinate at *d*, the result will be the average haul for all this material.

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If an area like *pmdnq* is measured by a planimeter, say in square inches, it can be rapidly converted into chain yards. For example, if in the mass curve 1 inch horizontal = 400 feet or 4 chain lengths, and 1 inch vertical = 10,000 cubic yards, then 1 square inch will represent 40,000 chain yards, which when multiplied by the number of square inches measured by the planimeter gives the total number of chain yards in the area of the loop.

Thus these areas between a base line or a balancing line and the mass curve are considered haul areas, as they are proportional to the haul of the material involved. Consequently in fixing a balancing line on a mass curve it is usual to attempt to reduce these areas as much as possible to get the most economic haul, provided it is not done at greater cost through borrow and waste.

Freehaul and Overhaul.—In the mass curve diagram the area *pmdnq*, expressed in chain yards, and divided by *dd'* in cubic yards gives the average haul for all the material *dd'* in 100-foot chain lengths. But it is usual when inviting tenders for railway construction work, especially in America, to fix a limit of what is called "freehaul," within which limit all material is hauled at the contract price, and beyond which an additional price is allowed for haulage. This additional price, which is generally an amount per cubic yard per 100 feet of haul, is termed "overhaul" price. If *tv* be the length of freehaul to the horizontal scale, and be placed in position with its extremities on the mass curve so that it is horizontal (this is easily done by marking off its length on a strip of paper and moving that strip, always kept horizontal, with one of its extremities on the mass curve until the other comes on), all the material between T (corresponding to *t*) and D will just fill the embankment between D and V, because *tv* is horizontal and its haul will be within the freehaul distance. But the material between P and T must be hauled to the embankment between V and Q, which is a distance greater than the freehaul. It is required to find the distance of the centre of gravity of the portion of cutting between P and T from the centre of gravity of the portion of embankment between V and Q. The most convenient way to do this is to measure the area *pmtt'* with a planimeter, and express the result in chain yards and divide this by the number of cubic yards in *tt'*. [Another useful way of measuring the area is by counting the number of squares, *i.e.* when the diagram is upon square-ruled paper.] The result of dividing this area by *tt'* is to give the horizontal distance of its centre of gravity from *tt'*.

With this length set out on a strip of paper, place it in position so that its extremities are respectively on the curve line *pmt* and the vertical line *tt'*, at the same time keeping it horizontal. In this way the line $\beta\alpha$ has been fixed and gives the distance of the centre of

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gravity of the cutting between P and T from the vertical line tt'. Similarly, $\gamma\delta$ gives the distance of the centre of gravity of the embankment between V and Q from the vertical line vv'. Therefore the horizontal distance between β and δ gives the total average haul for the material in the cutting between P and T. But this total length must be reduced by the distance of freehaul (tv) leaving the amount of overhaul distance. This overhaul distance is from Fig. 15, p. 74, $\alpha\beta + \gamma\delta$, and if it is expressed in 100-foot lengths and multiplied by the number of cubic yards in tt', it will give the number of chain yards of overhaul. Similarly, the portion xey has a horizontal dotted line of the freehaul length set in it, and will need the overhaul calculating in the same way as that already done. Between y and y the distance is less than the freehaul, but between g and z the same operation must be carried out.

Use of Centre of Volume in Calculating Haul and Overhaul.—In using the mass curve sometimes the centre of volume instead of the centre of gravity is used in calculating the distance of haul and overhaul, but as they rarely coincide it is not an accurate method. It involves less work, as it is then unnecessary to measure the area of the loop and divide the result by the ordinate, the horizontal line through the centre of the ordinate cutting the curve in the centre of volume. Thus in the mass curve diagram, p. 74, bisecting tt' and drawing a horizontal line through the point will give the centre of volume a little above β.

It is usual to tender prices for excavation and overhaul on cubic yards of material in place, *i.e.* before being excavated. But the mass curve is worked out for material corrected for the "swell constant." Accordingly, before making up the cost of the work, it is necessary to reduce the freehaul, and also the overhaul quantities to what they would have been if the material had not been corrected by the swell constant.

For example, suppose the freehaul volume in this case, as shown by the mass curve, was 9000 cubic yards of clay, and the overhaul 45,000 chain yards. Since the "swell constant" was 10 per cent., each 10 cubic yards only produced 9 cubic yards of embankment, and therefore each 9 cubic yards of embankment must be paid for as 10 cubic yards of excavation. Thus the 9000 cubic yards will be charged for as 10,000 cubic yards of excavation, and the 45,000 chain yards will be charged for as

$$\frac{100\ \%}{100\ \% - 10\ \%} \times 45{,}000 = \frac{100}{90} \times 45{,}000 = 50{,}000 \text{ chain yards}$$

Had the swell constant been a positive one instead of a negative one the quantities to be paid for would have been diminished instead of increased.

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Also, if the material in each cutting varied in its swell constant the amount of material and also overhaul due to the distribution of material for each cutting must be calculated separately.

Overhaul Methods.—The object of arranging the distribution of material in freehaul and overhaul quantities is to enable contractors and others to see quickly and accurately what excavation and haulage the work will involve, and thus to be able to submit a low tender.

The method of calculating overhaul should also be clearly stated.

The method shown on Fig. 15 is that approved by most engineers, but some engineers prefer to calculate the total average haul for all material in a cutting to be moved in the same direction, and deduct from that the freehaul length, and then multiply by the total material.

In both of these cases cuttings, from which all the material is tipped within the freehaul limit, are not included in the overhaul calculations.

A method of estimating overhaul which is sometimes adopted is that of calculating the average haul of all material in all cuttings, whether any of them are entirely within the freehaul limit or not, and deducting from it the freehaul length and then multiplying the result by the total material excavated. But this method is rather unreasonable and obscures the actual probable cost of the work.

Some engineers prefer to omit entirely the overhaul item, but for the reason mentioned above it is probably advisable to include it. It also throws more light on the distribution of material to be adopted.

The first method mentioned as being the most approved may seem to benefit the contractor by recording more overhaul in any specific case ; but where the competitive tender is prepared on the understanding that overhaul will be measured in this way, it should make no difference to him beyond the fact that he knows more clearly how his excavation and haulage is paid for.

Distance of "Freehaul."—It should be noted that the nature and extent of the work and the probable method of handling the material should determine the length of freehaul. If the amount of earthwork to be carried out is hardly sufficient to pay a contractor the expense of bringing heavy plant such as steam excavators, locomotives and wagons, or even the latter alone, upon the site, the freehaul should be short, say 500 to 1000 feet. If, on the other hand, locomotives are used a freehaul of less than 3000 or 4000 feet will probably prove inadequate. Thus if the excavation is heavy whilst the haul is comparatively short, the overhaul item is best omitted from the Schedule of Quantities.

Limit of Economic Haul.—Where material has to be hauled long distances the question naturally arises how far is it economical to do so. The answer is obtained from a simple calculation :—

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If it costs 1*s.* per cubic yard to excavate and tip material within the freehaul limit, and if it costs 2*d.* per cubic yard for spoil area upon which to tip this in the event of its being wasted, then the cost of wasting it will be 1*s.* 2*d.* Again, suppose it costs 1*s.* 6*d.* per cubic yard to borrow material at the end of the long haul, and that the freehaul limit is 500 feet, and the cost per cubic yard per 100 feet of overhaul is $\frac{1}{2}d$. Assuming l to be the limit of economic haul in chains or 100-foot lengths, we have :—

$$l \times \tfrac{1}{2} + 12 = 14 + 18$$
$$\therefore \tfrac{1}{2}l = 14 + 18 - 12 = 20$$

$\therefore l = 40$ chains or 4000 feet above the freehaul, the total haul in all being 4500 feet.

In this case no account was taken for swell constant, but where that is necessary the cost of a cubic yard of material when consolidated in the embankment in each case is what is required in the above equation.

Thus beyond this limit l, it is not economical to arrange for overhaul on the mass curve diagram. In such cases every effort should be made, with the help of the mass curve, to select suitable points at which to borrow and to waste, so as to reduce the haul which is within the economic limits by the greatest possible amount.

Mass Curve Problems.—Some interesting properties are liable to be overlooked in determining the most economic balancing line, taking into account l, the limit of economic haul.

(*a*) Fig. 16 gives a mass curve which shows all material balanced between *a* and *f*, but the distance *af* is greater than l. If a horizontal

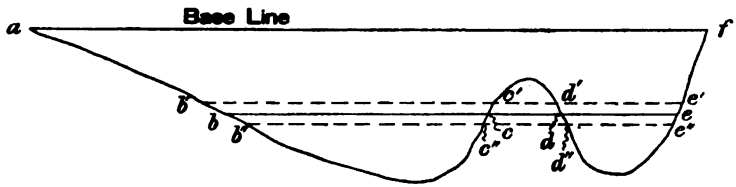

Fig. 16.

line *bcde* is drawn so that $bc + de = cd + l$, this is the economic balancing line.

Since *af* is greater than l, there is bound to be waste and borrow.

To show that the line *bcde* is the economic line :—

Suppose the line to be raised at *bcde* a distance equivalent to 1 cubic yard, then 1 cubic yard at *c* moves to *b* through the haul distance *bc*. Similarly, 1 cubic yard at *e* moves to *d* through the haul distance *de*, and the total haul for these = *bc* + *de* chain yards.

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Again, if the line is dropped, the cubic yard at c moves to d, and the cubic yard at b must be borrowed, and that at c wasted. But this borrow and waste is equivalent to l, and the haul from c to d to cd chain yards. But by hypothesis, $bc + de = cd + l$. Therefore, either method of dealing with the material will cost the same, which means that the line $bcde$ is the balancing line. This may be further seen by raising the balancing line to $b'c'd'e'$, the vertical height for 1 cubic yard when $b'c' + d'e' > c'd' + l$, as seen by the figure, thus making it cheaper to borrow a cubic yard for bb', and waste a cubic yard at ee', and haul a cubic yard from c' to d'. Similarly, by dropping the line to $b''c''d''e''$, the amount of 1 cubic yard, $b''c'' + d''e'' < c''d'' + l$, or it is cheaper to haul from c'' to b'' and from e'' to d''.

(*b*) Again, the fact that the area of any loop cut off by a balancing line is proportional to the average haul can be made use of in various ways in determining the most economic haul.

Fig. 17 gives a portion of mass curve corresponding to two cuttings with an embankment between them. The latter will only dispose of

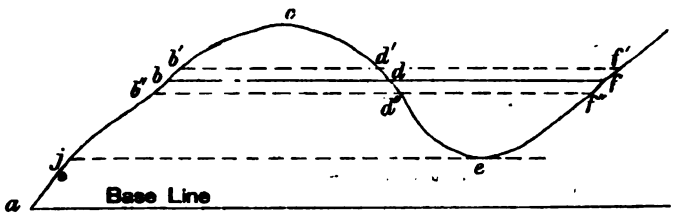

FIG. 17.

some of the material from the cuttings. Thus a trial balancing line may be made by drawing je horizontal and touching the curve at e. Then the average haul is proportional to the area $jbcde$. But if a line bdf be drawn cutting both loops, the sum of the areas divided by their depths will probably be less than that of one loop divided by the total depth. Now, if $bd = df$ the line will be found to be drawn in the best position for the least haul. This is best seen by trying the effect of the dotted lines above and below it at equal distances from it. Take the upper one $b'd'f'$ first: this line diminishes the area bcd by a strip of length less than bd, whilst the area def is increased by a strip of the same width but of greater length than df. Therefore the sum of the areas cut off by $b'd'f'$ are greater than those cut off by bdf.

Similarly, $b''d''f''$ increases the area bcd by a longer strip than that with which it diminishes the area def. Consequently the line $b''d''f''$ also cuts off two areas the sums of which for the same total depth are greater than $bcd + def$. Therefore, bdf, drawn so that $bd = df$ is a balancing line, which gives the least haul for a constant amount of material.

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(*c*) Similarly, if instead of two loops the mass curve have four loops, the line of least haul will probably be that which makes the sum of its first and third sections equal to the sum of the second and fourth. But if waste and borrow are no objection, the balancing line may sometimes be divided into two pairs each of equal lengths as above, and the borrow or waste occur between them where the lines have separated. If instead of separating the lines overlap, the solution becomes impracticable, and the single line cutting the four loops as above described must be used.

Thus, in studying the mass curve, the best distribution of the material and the cost of carrying out the work can be quickly and accurately found. It suggests the best places for borrow and waste, if these places prove satisfactory on the ground, and it points out very clearly any improvements which should be made or attempted on the profile, and is thus a guide to alterations in the located line which may prove advantageous. It is always felt that before making gradients steeper, a remedy should be attempted on the topographical plan without seriously spoiling the alignment.

Example from a Practical Railway Scheme

Fig. 18 shows a topographical plan of a short portion of the preliminary survey for a railway with the final location marked on it, and Fig. 19 gives the corresponding profile with final gradient line marked on, and above it the mass curve prepared for studying the distribution of its material, and showing the amount of overhaul.

This line is essentially a mineral line serving a coal mine area, the trains from which have to surmount the 1 in 250 or ·4 per cent. gradient with loaded wagons, and return the wagons empty or with light freight by hauling up a 1 in 100 or 1·0 per cent. gradient.

The dotted line shown on the plan is an approximate formation outcrop line for the gradients required. It was obtained by finding the surface point, at about every chain length, having the same level as the proposed gradient line at the point; and the system of points so found were joined by a smooth line which was put in dotted as shown. The line itself is not a true gradient line, as its length is much greater at some points than the located line.

A trial location was first made, and the best possible gradients fixed upon its profile, even though involving very heavy work. The dotted line was then located from this, and the final location on the plan made to follow the dotted line as nearly as good alignment and easy curves would permit.

This method of locating is a great help to the young engineer who has not had much practice at this class of work.

The ruling gradient, to the left, or in the direction of heavy haulage

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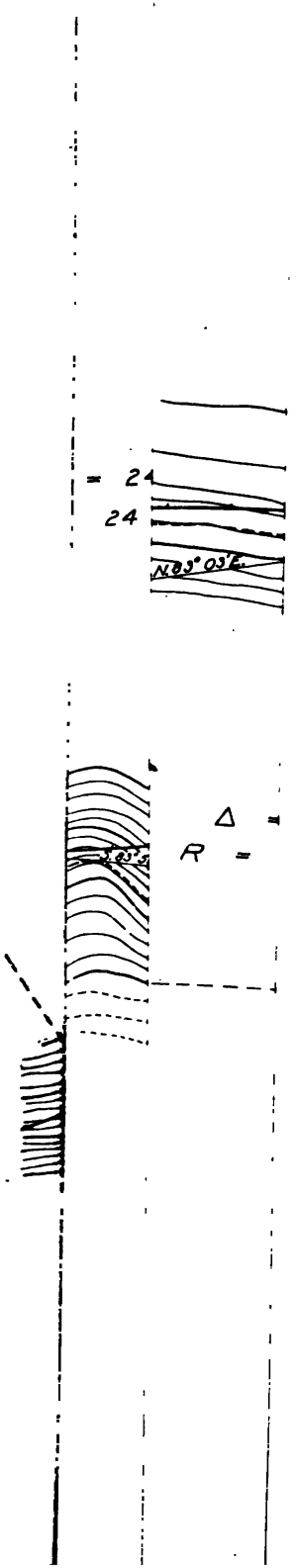

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was ·6 per cent., consequently the ·4 per cent. gradient used here allows plenty of scope for compensation on the 23·697 chains of 2°·122 curve. From the other direction the ruling gradient was 1·2 per cent., accordingly the 1 per cent. here located will enable the compensation on the 2°·865 curve between chainage 142 + 85·68 and 155 + 03·24 and on the 2°·387 curve between 159 + 83·79 and 170 + 28·12 to be easily applied.

For this railway the latitude and departure tables are given in Table V., p. 84.

Compensation for Curvature on Gradients

In studying haulage upon a railway, it is found that by far the greatest resistance to it is due to gradients. But at the same time that due to curvature cannot be overlooked, especially where the haulage is upon a ruling gradient or one nearly as steep. On such occasions any slight increase in the resistance may have the effect of overcoming the power of haulage.

Resistances on Curves.—The principal resistances on curves are those due to the degree of curvature, the length of the wheel base and the amount of wear to which the rails have been subjected. These latter, too, render the exact amount difficult to approach for general purposes, as nothing but an average for each railway concerned could possibly be taken. However, rules have been propounded for this resistance, and also for the amount of compensation which must be applied on gradients. A. M. Wellington suggested that the lowest limit of curve resistance at speeds not exceeding 30 miles per hour was about $\frac{1}{3}$ lb. per ton per degree of curve on good track, whilst with worn rails and rough track it might be as high as $\frac{2}{3}$ lb. But he also pointed out that this rule was not perfect as in the case of sharp curves [or with a high degree of curvature] the resistance so given was too great, whilst on flat curves of 1° or 2° of curvature the amount was too small.

Compensation.—When the line is on an incline, it is necessary to reduce the percentage of gradient in order to compensate for the resistance due to curvature. For this purpose it needs flattening as the case may require. Wellington suggested a reduction of ·05 per cent. per degree of curvature on gradients, and the rule is a good general one, but might be reduced to ·04 per cent. for degrees from 5° to 10°, and to ·03 per cent. for degrees above that. It is also possible to reduce these by ·01 per cent. on lines devoted to fast traffic. Again, where a curve is approached by transition curves so great a reduction is not needed. The locating of a line upon a long length of ruling gradient is a very difficult problem on account of the necessity for this compensation.

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TABLE V.

Latitudes and Departures of Preliminary Line.

From station.	To station.	Angle.	Distance.	Bearing.	N.	S.	E.	W.	Totals. N.	Totals. S.	Totals. E.	Totals. W.
	143+20								259·3		13906·3	
L. 143+20	M. 151+01·5	21° 59′ 20″ R.	781·5	N. 65° 40′ 30″ E.	321·9		712·1		580·2		14618·4	
M. 151+01·5	N. 162+76	8° 28′ 30″ R.	1174·5	N. 87° 39′ 50″ E.	48·1		1173·5		628·3		15791·9	
N. 162+76	O. 171+50·5	23° 49′ 00″ L.	874·5	S. 83° 51′ 40″ E.		93·6	869·5		534·7		16661·4	
O. 171+50·5	P. 178+31·4	6° 30′ 00″ R.	680·9	N. 72° 19′ 20″ E.	206·8		648·8		741·5		17310·2	
P. 178+31·4	Q. 188+50·9	4° 13′ 40″ R.	1019·5	N. 78° 49′ 20″ E.	197·7		1000·1		939·2		18310·3	
Q. 188+50·9	R. 198+72·5	29° 31′ 40″ R.	1021·6	N. 83° 03′ 00″ E.	123·6		1014·1		1062·8		19324·4	
R. 198+72·5	S. 206+65·1		792·6	S. 67° 25′ 20″ E.		304·3	731·8		758·5		20056·2	

Latitudes and Departures of Proposed Location.

Sub-Tang.	E.C.	Tangent.	B.C.	Sub-Tang.	P.I. to P.I.	Bearing.	N.	S.	E.	W.	Totals. N.	Totals. S.	Totals. E.	Totals. W.	Curve.	Angle.	Radius of curve.
											214·3		13904·7				
		476·6	142+85·7	628·4	(1165·0)	N. 62° 21′ E.	512·0		979·6		726·3		14884·3		2°·865 R.	34° 53′	2000′
628·4	155+03·3	480·5	159+83·8	530·6	1639·5	S. 82° 46′ E.		206·4	1626·4		519·9		16510·7		2°·3875 L.	24° 56′	2400′
530·6	170+28·1	799·8	178+27·9	1268·6	2599·0	N. 72° 18′ E.	790·2		2476·0		1310·1		18986·7		2°·12 R.	50° 20′	2700′
1268·6	201+99·7				(1259 0)	S. 57° 22′ E.		678·9	1060·3		631·2		20047·0				

The figures in brackets give distances from an initial or final point.

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Example.—The following example will illustrate this. Take the ruling gradient at 1 in 100, or 1·0 per cent., and the distance between two points connected by the ruling gradient at fifty stations or 5000 feet, the difference in elevation of the two points being 50 feet. Let it be supposed that it is necessary to have eight stations (100 feet each) of 3° curve, and seven stations of 2° curve. Then, allowing for compensation for curvature as above, the total gain in elevation will be—

35 stations	at 1·0 %	=	35·0 feet
8 „	at (1·0 % − 3 × ·05 %) or ·85 %	=	6·8 „
7 „	at (1·0 − 2 × ·05 %) or ·90 %	=	6·3 „
	Total	=	48·1 „

But the difference in elevation of the two points is 50 feet, consequently even upon the ruling gradient the elevation has not been surmounted by nearly 2 feet.

Supporting the Line.—Thus it is necessary to adopt some other method of supporting the line so as to develop it still more. But in thus gaining additional length the amount of curvature, which is almost bound to be increased, should be kept down as low as possible.

In changing the location of this line the method illustrated by the dotted line in Fig. 18 will prove very useful. It frequently happens that long lengths of straight can be located at little cost on a gradient flatter than that really required to reach a limiting point, and that when leaving the straight the curves are naturally upon a steeper gradient. Such location is fatal to satisfactory results. Every effort should be made to locate the straights on the steeper gradient, thus enabling that on the curves to be flatter, and so increase the efficiency of the line.

Vertical Curves

Vertical curves are used at points of change of gradient to round off the gradient line and prevent sudden shocks to trains, and the tendency to break couplings.

Circular curves have occasionally been used for this purpose, but the correct one, and that more easily set out, is the parabolic curve, which is now in common use for this purpose. On account of the properties of a parabola, uniform changes in the percentage of gradients at equal intervals along the gradient line will produce the curve.

Examples.—The sag (*i.e.* where the gradient lines intersect in an obtuse angle above) ADG in Fig. 20 is formed of two gradients of + ·8 per cent. and − ·4 per cent. intersecting at D.

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On American railways at changes of gradient, at either sags or summits, of ·3 per cent. and upwards, it is found necessary to use vertical curves. The rate of change of gradient per station (100 feet) on these curves is an amount less than ·3 per cent.

Thus in Fig. 20 the change in gradient at D as given by deducting the one from the other is—

$$+ \cdot 8 \text{ per cent.} - (- \cdot 4 \text{ per cent.}) = 1 \cdot 2 \text{ per cent.}$$

And taking a rate of change per station of ·2 per cent., it will require $\frac{1 \cdot 2}{\cdot 2} = 6$ stations for the length of the curve. To find the tangent points A and G, measure 300 feet in each direction from D, and let A, B, C, D, E, F and G be the station points. Then, the level

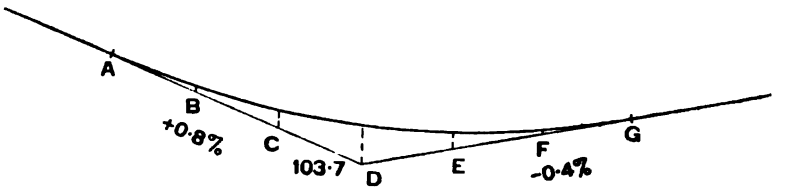

FIG. 20.

of D being 103·7, those of the other points A, B, etc., can be calculated They are given in Table VI.

TABLE VI.

VERTICAL CURVE TABLE.

Station point.	Change of rate of radient per 100 feet.	Rate of gradient.	Reduced level of points on gradient lines.	Reduced level of corresponding points on curve.
		+ ·8%		
(B.C.) A	·1%	+ ·7%	106·1	106·1
B	·2%	+ ·5%	105·3	105·4
C	·2%	+ ·3%	104·5	104·9
D	·2%	+ ·1%	103·7	104·6
E	·2%	+ ·1%	104·1	104·5
F	·1%	+ ·3%	104·5	104·6
(E.C.) G		+ ·4%	104·9	104·9

In the second column in the table it may be noted that the change of rate of gradient between A and B is ·1 per cent.; this is due to its being the average between the rate of change at A (·0 per cent.) and at B (·2 per cent.). The same occurs between F and G. From the fifth column the level of the curve above D is ·9 feet.

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Fig. 21 represents a vertical curve at a summit. It is usual to make the length of the curve in summits half of that which it would be in a sag for the same rate of change of gradient. But the length is always made an even number of 100-foot stations. The Canadian Pacific Railway makes the length of the vertical curves in sags as many stations 100 feet long as there are ·1, or one-tenths in the change of the gradient rates. Thus in above example the curve would be $\frac{1{\cdot}2}{{\cdot}1}$ stations = 1200 feet in length. Also in summits the length would be $\frac{1{\cdot}2}{{\cdot}2}$ stations = 600 feet. The change of rate of gradient as given in the second column of the table would be ·05 per cent., ·1 per cent., etc.,

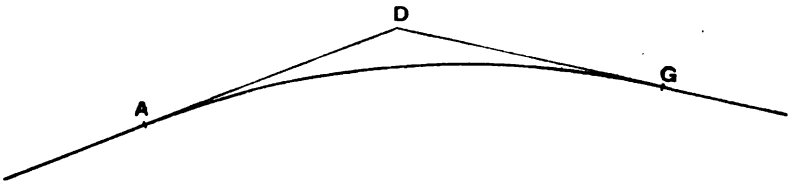

Fig. 21.

in sags and ·1 per cent., ·2 per cent., etc., in summits. Thus the figures in the table would have done for the case of a summit on the Canadian Pacific Railway.

Selecting and Fixing the Positions of Vertical Curves.—The need for vertical curves is more felt in the case of low speeds than of high speeds, the length of them in the latter case being made shorter or omitted altogether.

Location is not considered good which places a sag in a cutting or a summit on an embankment. The sag on an embankment requires additional filling when rounded off by a vertical curve, and in the case of a summit in a cutting more excavation is required if this refinement is applied. Consequently vertical curves generally mean additional earthwork.

The setting out of vertical curves is simply the operation of taking levels on the station pegs, and finding at each station point the difference in level of the peg and the formation or subgrade as given by the corresponding figure in the fifth column of the table.

Momentum Gradients

The kinetic energy possessed by a train travelling at a uniform velocity can be utilized in overcoming resistances greater than those experienced in mounting a ruling gradient. In locating a line, more

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especially where heavy cost of construction must be avoided, gradients greater than the ruling gradient are sometimes adopted. But their adoption can only occur where it is possible for the train, when entering such a section of steeper gradient, to have stored in it sufficient kinetic energy to enable it to overcome the additional resistance due to the excess of steepness of the gradient over and above the ruling gradient. This kinetic energy is usually acquired by approaching the steep up-grade on a down-grade. These gradients, steeper than the ruling gradient, are called momentum gradients. The velocity heads of trains acquired in descending various depths on down gradients was carefully investigated by A. M. Wellington, who assumed a 6·14 per cent. loss of total energy due to the rotation of the wheels in preparing the data in Table VII., thus making the head for any given velocity greater than that which would be obtained by using the formula $h = \frac{v^2}{2g}$ where h is the head in feet, v the velocity in feet per second, and g the acceleration of gravity.

TABLE VII.

TABLE OF VELOCITY HEADS.

Miles per hour.	Velocity head in feet.	Miles per hour.	Velocity head in feet.
10	3·55	26	24·00
11	4·30	27	25·88
12	5·11	28	27·83
13	6·00	29	29·86
14	6·96	30	31·95
15	7·99	31	34·12
16	9·09	32	36·35
17	10·26	33	38·66
18	11·50	34	41·04
19	12·82	35	43·49
20	14·20	36	46·01
21	15·67	37	48·60
22	17·19	38	51·26
23	18·79	39	54·00
24	20·46	40	56·80
25	22·20		

The 6·14 per cent. above mentioned was an average; for trains heavily laden it was 1 or 2 per cent. less, whilst for empty trains it was sometimes 4 or 5 per cent. higher.

What is generally understood by a ruling gradient is one which must be approached by a train with a minimum velocity of 10 miles per hour or a velocity head of 3·55 feet (see table). Consequently, if at the end of a momentum gradient a ruling gradient has to be surmounted, sufficient momentum must be retained to provide a velocity of 10 miles

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per hour upon entering the ruling gradient, thus reducing the maximum length of momentum gradient.

Example.—The following example will illustrate this (Fig. 22.) A train approaches from A down an easy gradient to B, where its velocity is 30 miles per hour. At this point a momentum gradient of 1·2 per cent. begins and runs to C, where it changes to the ruling gradient ·8 per cent. It is necessary to find the length of momentum gradient BC which can be located. The table gives the velocity head at B as 31·95 feet. Since CD is on a ruling gradient, the velocity head at C must be

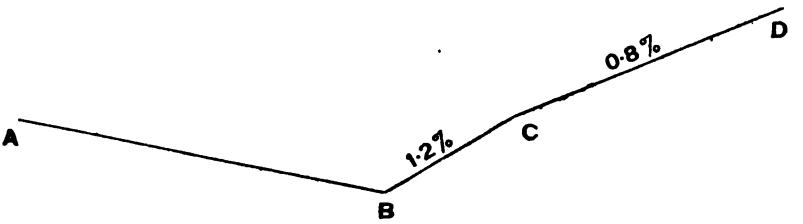

FIG. 22.

3·55 feet. Therefore the velocity head which can be lost between B and C = 31·95 − 3·55 = 28·40.

But the engine can haul upon a ·8 per cent. gradient. Therefore it will overcome the part of the resistance due to this gradient and leave the balance due to 1·2 − ·8 = ·4 per cent. to be overcome by the momentum. Thus the momentum is used up on what might be called a ·4 per cent. gradient.

Therefore the length of BC is given by 28·4 ÷ ·4 = 71 stations = 7100 feet. By the inverse process the velocity, which must be attained at B relative to a definite length BC, may be calculated.

Objections to Momentum Gradients.—The objections to momentum gradients are several. No stopping can occur on the gradient AB if its length is only sufficient to raise the velocity at B to the required amount.

The gradient AB must not be as steep as the ruling gradient, since in the event of the train being stopped by any cause near the bottom of the sag it will not be able to move either way. The sudden changes of speed put severe strains on the couplings and on the cars themselves in sags.

Use of Table.—In using the table of velocity heads for purposes of momentum gradients, it will be advisable to increase the velocity heads 25 or even 50 per cent. where some intimate knowledge of the resistances in rolling stock is not obtainable. It must also be

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remembered that a new track offer much more resistance to traction than one which has become smooth by constant usage.

The Virtual Profile

The virtual profile is the line joining the points, representing the velocity head, set off from each point on the ordinary profile for any selected initial velocity.

Example.—The following example will illustrate this. (Fig. 23.)

It is assumed that the train reaches A with a velocity of 25 miles per hour, and that the pull of the engine is just sufficient to overcome all resistances, and thus preserve a uniform velocity on the level. If the engine continues to exert the same pull on the down gradient AB the velocity will increase due to gravity. Thus at A the velocity being

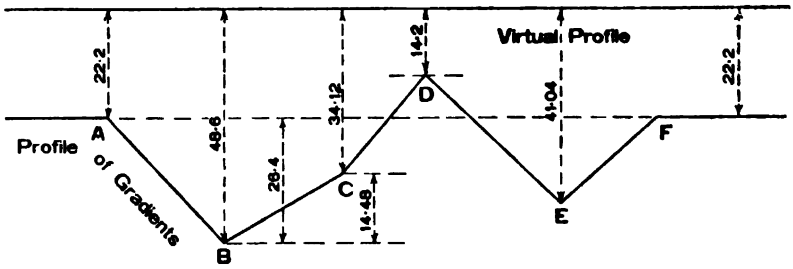

Fig. 23.

25 miles per hour or equivalent to a velocity head of 22·2 feet, the line of virtual profile at this point will be 22·2 feet above A. Again, when the train reaches B, which is 26·4 feet lower than A, its velocity head will be 26·4 + 22·2 = 48·6 feet, which from the table will be seen to correspond to a velocity of 37 miles per hour. If the engine continue to exert the same pull in running from B to C, the train will have mounted 14·48 feet, thus reducing its velocity head to 34·12 feet, and its velocity to 31 miles per hour. Again, at D the velocity head is 14·2 feet and the velocity 20 miles per hour; at E it has increased again to 41·04 feet, and the velocity to 34 miles per hour; and lastly at F the head is again 22·2 feet, and the velocity 25 miles per hour as at A. Thus the system of points above A, B, C, etc., corresponding to the velocity heads, lie on a horizontal line called the virtual profile. By a study of the virtual profile line the position of which has been established by an assumed initial velocity, the velocity at each point of the located profile can be determined by scaling off the velocity head from the virtual line, and finding the corresponding miles per hour from the table of velocity heads. If the point D had been on the virtual profile,

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the velocity head would have been zero and the train would have come to rest. Thus it is necessary to have all points on the profile below the virtual profile. From Fig. 23 it is seen that so long as the engine exerts a uniform pull, sufficient to keep the velocity constant on the level, the undulations in the road bed as shown on the profile will not affect the hauling demands on the engine, but will eventually bring the train back to the same level at F as it was at A, with no loss in velocity. [This, though a theoretical condition is found in practice to be approximately correct, some slight modifications are at times required due to imperfections in the track, and to variations in resistances to hauling on up and down grades and on the level.]

Slope of the Vertical Profile.—Virtual profiles are not always horizontal. If, in hauling a train upon a gradient, the engine preserves a uniform velocity due to its exactly overcoming the ordinary resistances together with that due to gravity, the virtual profile is parallel to the gradient line, and at a vertical distance from it equal to the constant velocity head. To have the virtual profile parallel to the gradient line on a down gradient would require the resistance of the brakes added to the ordinary resistances to equal the effect of gravity, and thus preserve a uniform velocity. Thus we see that the virtual profile is parallel to the gradient line when the velocity is uniform; that the velocity is increasing when the profile lines are separating and diminishing when they are approaching each other; and that when they coincide the velocity is zero.

Use of Virtual Profile in selecting Gradients.—In order to study the effect of gradients at points where hauling may be difficult, virtual profiles may be constructed for both fast and slow traffic, if necessary, but that for the latter is generally sufficient.

If a portion of road was designed for operating according to a virtual profile and at some subsequent period a station or stopping place was located between its extremities, it may be found impossible to operate the road without an additional engine. But in any case of this kind the virtual profile will point out a remedy if such is possible.

An ideally located line is one on which an engine will haul with a constant pull throughout its length. But no such location can be performed without an intimate knowledge of the locomotives, rolling-stock and sizes and weights of trains to be used on the line.

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CHAPTER VI

FIELD LOCATION

When the line is finally located in the office on the topographical plan and the corresponding profile, with gradients marked on, is prepared together with the material distribution curve, the next operation is to reproduce it on the ground. In performing this reproduction it must be remembered that it is a line with the same properties, profile, etc., or a better one, which has to be located on the ground. It can hardly be expected that the topography on the original topographical plan, prepared during the preliminary survey, will have been located so accurately with reference to the preliminary traverse, as to ensure that the line, which is an accurate reproduction of the office location with respect to the traverse line on the ground, will have the identical properties looked for. But what generally happens is that the line obtained in this way is very near the required one and only needs adjusting on the ground. Consequently, in transferring the office-located line to the ground, any extremely accurate method of performing this work is a waste of time, as the line will invariably require readjustment on the ground. However, there is no need for carelessness or adopting haphazard methods when simple and at the same time rapid methods can be adopted.

Setting Out the Office Location on the Ground.—The following is the general method adopted and applies to most parts of a line over any terrain. Fig. 24 shows the traverse ABC, etc., and two tangents or straights of the located line A′B″E″G″ as produced in the office on the topographical plan. The traverse ABC, etc., is already staked out on the ground, being the preliminary line. The positions of A′, B″, E″, and G″, can be carefully scaled on the plan and later marked out on the ground according to their scaled chainages. For instance, if a theodolite be set up at B and sighted on A, A′ can be placed truly on line. Similarly by sighting on C, B″ may be accurately located.

This instrumental work is frequently necessary, as only some of the stations between turning points are lined in by the instrument during the preliminary survey. As only a few points may be obtained by

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intersections like A′, B″, etc., it is necessary to draw in square offsets to the traverse lines on the plan and to scale off their lengths to the located line as at B, C, D, E, F, G, and H. These offsets should each be made square to that traverse line which is most nearly parallel to the located tangent. Thus the offsets at both C and D are at right angles to the line CD as being more nearly parallel to the tangent than either CB or DE. Again, at E the offset is at right angles to EF which is nearly parallel to A′B′C′D′. Sometimes it is necessary to take offsets at station points between turning points on the preliminary traverse, where traverse lines are long or the country is wooded. In such cases the station points used must be lined in by the instrument. These offsets are carefully scaled on the plan. The most practical method of conveying the information of lengths of offsets, etc., into the field is on a tracing or sketch of the lines on the plan, each offset having its length marked upon it. Also the chainages of such points as A′, B″, E″, etc., should be marked on.

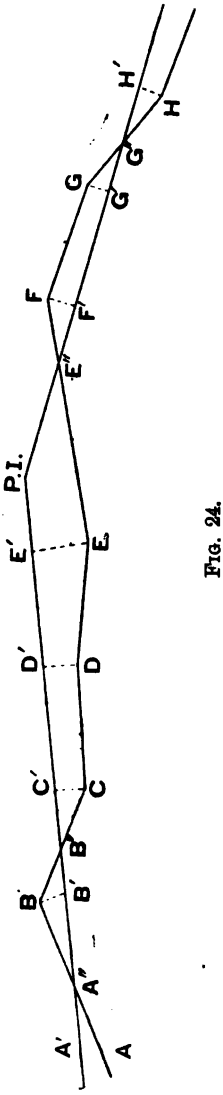

FIG. 24.

With sufficient data of this kind for a day's work and also with the office profile or a tracing of it, the locating party will take the field. Points such as A′, B″, etc., are taped out and fixed in position, ranging poles being set up at them. If the offsets at B, C, D, and E are short, not more than 100 or 150 feet and the ground is level or nearly so, they may be squared off with an optical square and the position fixed by taping. If, however, the ground slopes considerably a cylindrical cross-head with plumbing adjustment may be used satisfactorily for squaring off. It must be remembered that the tendency to error in the position of the point located is much greater if the traverse line from which the right angle is being set off is not nearly parallel to the location tangent. In such cases it is generally more satisfactory, especially on sloping ground, to set off the right angle by the theodolite. At each of the points on one tangent located as described ranging poles are set up. The number of points fixed in this way on any one tangent should not be less than 4 and preferably 6 or 7,

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and more if they cannot all be seen from either end. These ranging poles ought to lie approximately in a straight line. But an error of a few feet on one side or the other must be expected, as the scale of the plan on which they were measured may have been as fine as 400 feet to 1 inch where the thickness of a pencil line would amount to a few feet.

The ranging poles are now adjusted by eye to occupy a straight line. The skilled locator does this quite quickly, striking the mean position of the poles and partly ignoring any which seem to have an excess of error.

Testing the Levels on the Set-Out Lines for agreement with the Location Profile.—It is now advisable to test the levels at a few points on this line or at several if it is a long one. This may be rapidly done with a hand-level, obtaining the collimation or level of the eye by sighting on the staff held at the section point on the preliminary traverse. It is generally most convenient to test the surface levels at such points as A′, B′, etc., at which ranging poles are standing and the approximate levels of which can have been marked on the tracing in the office. But sometimes other important points exist, such as summits or bottoms of depressions, which require checking also. An error of a foot in level either way and more in the case of very rough ground must be ignored in this operation. If, however, all points tested are consistently high or low, it is necessary to find the approximate location of the line to the right or left of that polled out, whose surface levels more nearly agree with those on the profile of the located line. This can be done by fixing a few additional ranging rods at the points of approximately correct level, the levels being taken by the hand-level at the same time as the line levels are being tested. This operation can be done very quickly and need not be any source of delay. But a little care in locating the correct position of the line well repays the labour. On this account a good number of points must be prepared in the office, lest they fail to be located on account of obstructions. If the line is shifted to the right or left, its new position should be sketched on the tracing and three or four points in it fixed by swing-offsets from the traverse line.

Fixing the Points of Intersection.—When two consecutive tangents of the located line are ranged out in this way their point of intersection, the P.I., is sighted in so as to line with both of them. A large peg 2 to 2½ inches square is then driven at the P.I. and centred with a small flat-headed nail. The instrument is now set over this point and the previous P.I. sighted upon.

Curves and their Tangent Points.—If the next P.I. has also been fixed the deflection angle between the two tangents can be read and the positions of the tangent points of the curve calculated and fixed on the ground, as the degree of curvature is already given on the plan. The

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tangent points are fixed by driving large pegs as above and centring them with a nail. They are further marked by driving two smaller-sized pegs (or guard pegs) one on each side of the line about 6 inches from the centre and with their points sloping in towards the centre peg. The curve may or may not be run in on the ground now, according to whether the party is deputed to do that part of the work or not. If so the chainage of the previous part must have been complete up to this point.

In mountainous country curves instead of tangents may predominate, in such cases and also in the case of long curves in other country, points on the curves must be located by the offset method and checked for level in order to compare with the levels on the profile, as the tangents are too far removed from the curves at points near the P.I. But the P.I.s for all curves must be fixed also to enable the curves to be accurately set out later.

Locating a Long Length of Straight on the Ground.—In open, level country, where probably location is performed without any preliminary survey having been made, the problem of locating a long level of straight has frequently to be confronted. The most satisfactory way, if possible, is to direct the telescope of the instrument upon a distant object on the line to be followed, then fix between 10 and 20 stations on this line, carefully centring the peg driven at the last one, before moving the instrument to it as the next observation point. The instrument, when set accurately over this new point and levelled up, will have its telescope directed immediately on the distant object without any backsighting and the next sequence of stations will then be established. The only object for which backsighting would be useful is as a check on the accuracy of the instrument, thus showing immediately if the collimation line is truly at right angles to the axis of rotation of the telescope. But it is not necessary to have this in adjustment for setting out a straight by the distant object method. If no distant object can be sighted upon, the method of producing a straight line explained on p. 46 must be adopted.

Chaining the Located Line.—When a sufficient number of P.I.s have been located and possibly the B.C.s and E.C.s (the beginnings and ends of curves) the line must be carefully chained out. Each station point must be lined in by the instrument, the points being marked by solid 2-inch square pegs driven flush with the surface of the ground. On curves, where the curvature does not exceed 4° or 5°, if the station points are 100 feet apart, intermediate pegs must be fixed at the 50-feet points. On curves sharper than 5° it is advisable to have the points closer together than this amount, 25-foot intervals being frequently used. If the gunter chain (66 feet) is being used, the interval it affords is

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sufficiently small up to about 4° curves (21·7 chains radius). Probably a better division point would be at the 20 chains radius curve. For all curves with smaller radii 33-foot intervals should be used.

The marking of chainages on the located line should be done as in the case of the preliminary line on small labels of wood fixed in the ground to the right of the pegs. The marking should be done in a distinguishing colour. Thus if the preliminary traverse is marked in blue the location may be marked in red.

Fixing Reference Pegs for the Preservation of Important Points.—The method of running in the curves will be described presently. When each curve is fixed the tangent points, or B.C. and E.C., have their position carefully recorded by driving reference pegs beyond the excavation or embankment on each side of the line.

Fig. 25 shows a common method of doing this. When the instrument is set up at the tangent point B.C. for the purpose of setting out

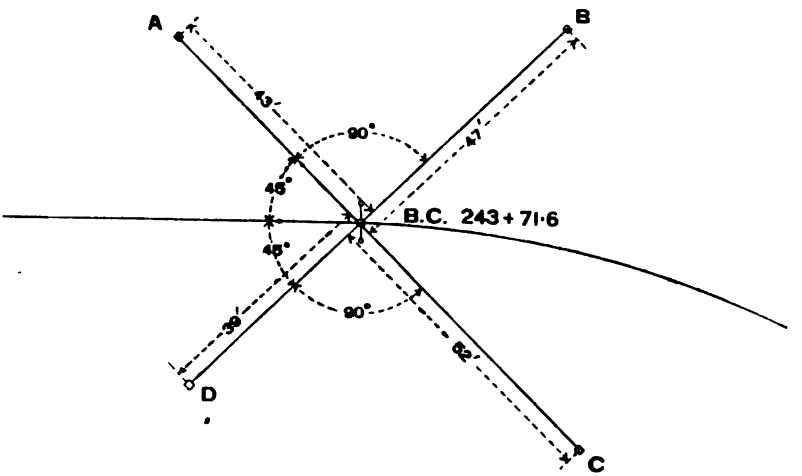

FIG. 25.

the curve, a line to A can be set off by making 45° with the tangent and the point A can be chosen so as to be quite clear of the earthwork. A peg is then driven at this point and its distance (43 feet) from the B.C. taped. The telescope is now transited and a corresponding point C fixed on the other side. Then turning the angle through 90° D and B may be fixed. It is not necessary to set off 45° and 90° as shown, but the best results are obtained by fixing the lines AC and BD at right angles to each other. By thus referencing important points on the located line, they can be reset accurately in their horizontal positions when the earthwork is completed, by setting up a theodolite at one of

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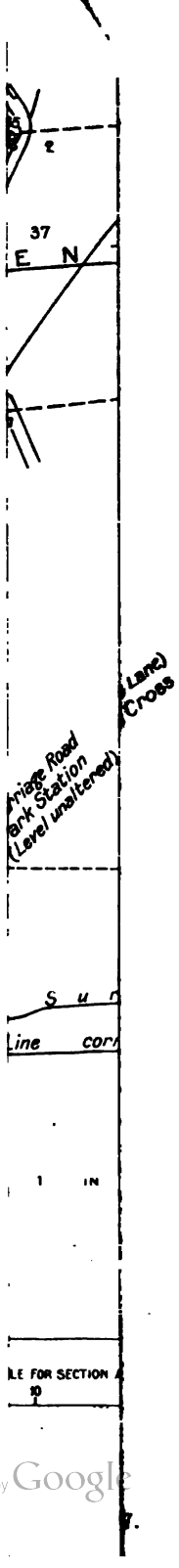

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the points as A and sighting on C and putting in a peg at 43 feet from A, and can be checked for centre by measuring the distances from B and D.

Completing the Location Survey and Plan.—When the line is chained out a special survey should be made of all boundary lines and other artificial topography through which it passes. If a theodolite is not being used to fix the bearings of such lines as cut the located line, their directions may be determined by the box sextant. The names of all owners of property are usually entered upon this final plan, in the areas owned by them (see Fig. 18). All river and road crossings are accurately surveyed. In this country deviation lines 300 feet on either side of the centre line are shown on this plan (see Fig. 26). When authority is obtained by Act of Parliament to construct a line, power is granted to vary the position of the centre line within these limits. But it is necessary to show on the plan a complete survey of the full width between deviation limits. The railway boundary lines or limits of right of way corresponding to the centre line shown must also be plotted on. This is obtained from the calculated half-widths of cuttings and embankments together with the lengths from the edge of the slopes to the boundaries. The lines formed by these widths in rapidly undulating country are so wavy that it is generally necessary to straighten them on the ground by conceding as much land at one place as is taken in at another. Where borrow pits or spoil banks are needed additional widths must be provided for.

The Levels and Final Profile.—The levels of the located line must be taken very carefully, bench-marks being established about every half mile. Levels should be taken on every station peg as well as at any intermediate points where changes in the surface occur, in order to be able to plot an accurate profile of the line. Levels may be checked upon bench-marks or turning points of the preliminary traverse at points where the two lines approach each other.

The profile when plotted will have the gradient located upon it. This should be plotted in pencil pending its final confirmation. At this stage, or whilst the levels are being taken, borings should be made at various points in the cuttings to show the nature of the material to be excavated. Where alternations in the depths of rock and clay exist at any points, it is advisable to take sufficient borings to enable the best position for the line to be found which will be most economical in construction whilst conforming with other requirements.

Checking the Profile on the Ground and making the Final Revision.—Taking this profile and the boring information with him on to the site, the locating engineer is enabled to make further refinements in the line, which will become more evident when studied thus on the

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ground together with the information provided by the profile. He will also observe the best places for borrow pits and thus determine how the material is to be distributed. Whilst walking over the located line in this way he will observe the river crossings and decide upon the size of spans and nature of bridges, culverts, etc.

The profile is now taken back to the office and the gradient line which is pencilled upon it again studied and any revision which seems advisable made in it. The effect of this revision on material and its distribution should also be investigated on the "distribution curve" and an estimate prepared which, if found satisfactory, enables the inking in of the gradient line to be performed. Sites for bridges and other structures should be surveyed more accurately and plotted to a larger scale, 20 to 40 feet to an inch.

These surveys should include abundant levels of the surface, also showing the rate of fall of the river or road. Also high and low water levels should be obtained. The former are generally obtained along river valleys marked by the deposit of brushwood and river debris. Where the highest layers thus deposited on the river slopes are taken, it is usually assumed to be a fair indication of high water.

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CHAPTER VII

LOCATION

Simple Railway Curves

The circular curve is that commonly used in railway work to connect lengths of tangents or straights together. Although the sudden change in motion due to leaving a straight path for a circular path has caused much discomfort to passengers, as well as risk of accident to trains, the circular curves still hold their place, though in most cases now the approaches to them are eased by transition curves.

Expressions for Curvature.—There are two ways of expressing the curvature of any railway curve :—

(1) In Great Britain it is referred to as of a certain number of chains radius, and as the chain used is the Gunter chain (66 feet long) the curvature is definitely fixed by so describing it.

(2) In America and other countries the degree definition is used for the curvature. Thus a 4° curve is a portion of a circle of such radius that 1 chain chord subtends at the centre of the circle an angle of 4°. In Fig. 27, D° corresponds to 4°, being the degree of the curve. In America the chain chord used is not 66 feet but 100 feet.

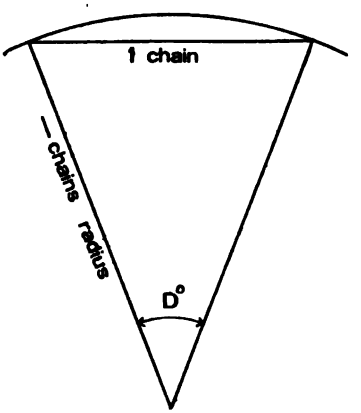

Fig. 27.

But it should be noted where any doubt exists as to the length of chain used the curvature is not defined by stating the degree of the curve, as a 4° curve "set out" with a 66-foot chain is much sharper than when "set out" with a 100-foot chain.

In dealing with (1) it is necessary to make use of the radian (the unit of circular measurement). This which is the angle at the centre

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of a circle subtended by an arc equal to the radius, is equal to 57·3° approximately. In the case of railway curves the lengths along the circumference of the circle are measured with chords generally 1 chain long; consequently there is a very slight error in making use of the radian, as the arc corresponding to 57·3° at the centre will, when measured by chain chord lengths, give a less total than the chains in the radius, but only very approximately less. This is evident, as a chord 1 chain long will deviate very little from the arc in the case of circles of over 10 chains radius, and few railway curves are sharper than these except in terminal stations, where speeds are very small, or in special cases in mountain railways.

$$\frac{57°{\cdot}3}{\text{radius in chains}} = \text{number of degrees subtended by 1-chain chord}$$

This, which is called the degree of the curve, is designated by the letter D.

$$\therefore \frac{57°{\cdot}3}{\text{radius in chains}} = \text{D}°$$

In setting out a circular curve of D degrees, the deflection angle for each chain length is $\frac{1}{2}$D°. (This will be shown presently.)

Now, 57°·3 = 3438′

$$\therefore \frac{3438}{\text{radius in chains}} = \text{the degree of the curve in minutes}$$

But half the degree of the curve is the deflection angle for 1 chain length.

$$\therefore \frac{1719}{\text{radius in chains}} = \text{deflection angle in minutes}$$

This is the formula commonly used in setting out railway curves in Great Britain. The more accurate value of 1719 is 1718·87338, and the value 1718·9 is often used instead of 1719. Many engineers make use of this formula to determine their deflection angles without knowing anything of its origin.

In "running in" railway curves, the line "set out" by the instrument is usually placed in the centre of the formation. Thus a single-line track will have its centre line given in the middle of the track between the rails. In the case of a double line, the centre line will be fixed in the middle of the 6-foot way, and equally distant from the two roads.

In Great Britain engineers dealing with the question of curvature on railways think in chains' radius. Thus a 15-chain curve with the usual superelevation would be considered too sharp where a speed of 40 miles per hour was required, a 20-chain curve, or one even flatter than this being considered advisable. In this way the relative values

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of the various curves are thought of and expressed. The American engineer in a similar way thinks of his curves in degrees with reference to the permissible maximum speeds of trains to be run on them.

Curvature Expressed in Degrees or Radius in Chains.—The relation between the degree of a curve and the radius of curvature are clearly seen from Fig. 28. The line joining the centre point of the 100-foot chord to the centre of circle, being at right angles to the chord, affords the following properties :—

$$\sin\left(\frac{D^\circ}{2}\right) = \frac{50}{R}$$

$$\therefore \text{R (the radius of curvature)} = \frac{50'}{\sin \frac{1}{2}D^\circ} \text{ feet} \quad . \quad . \quad . \quad (a)$$

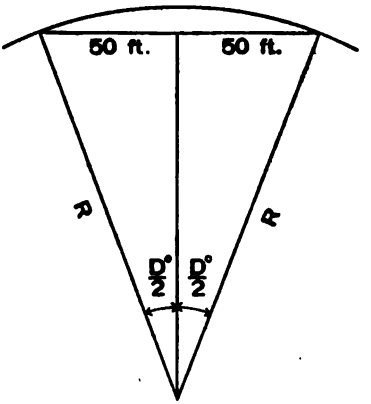

Fig. 28.

An approximate value for the R may be found by replacing sin $\frac{1}{2}D^\circ$ in this formula by $\frac{1}{2}D^\circ$ in radians.

$$\text{Thus R} = \frac{50}{\frac{\frac{1}{2}D}{57{\cdot}3}} = 50 \times 2 \times \frac{57{\cdot}3}{D} = \frac{5730}{D} \quad . \quad . \quad . \quad (b)$$

This is only useful in the case of small values of D, or for the purpose of obtaining approximately the value of R.

The Elements of a Railway Curve.—The ordinary problem met with in the field in running in railway curves is that of connecting two tangents or straights by a circular curve, the degree of which has been decided upon.

Thus in Fig. 29 a portion of two located tangents B.C. P.I. and P.I. E.C. is shown, intersecting externally in angle $\triangle$ at P.I. (the

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point of intersection). These require to be connected by a circular curve, and it is assumed the required curve is B.C. E E.C.

B.C. (the beginning of curve) and E.C. (the end of curve) are the points where the tangents touch the circle. Therefore B.C. P.I. and E.C. P.I. are the common tangents from P.I., and are consequently equal. Each of these is called T. T is commonly called the sub-tangent, the term tangent being applied to the length of straight connecting any two circular curves.

The central angle B.C. O E.C. is equal to the external angle Δ because the tangents forming the latter are respectively at right angles

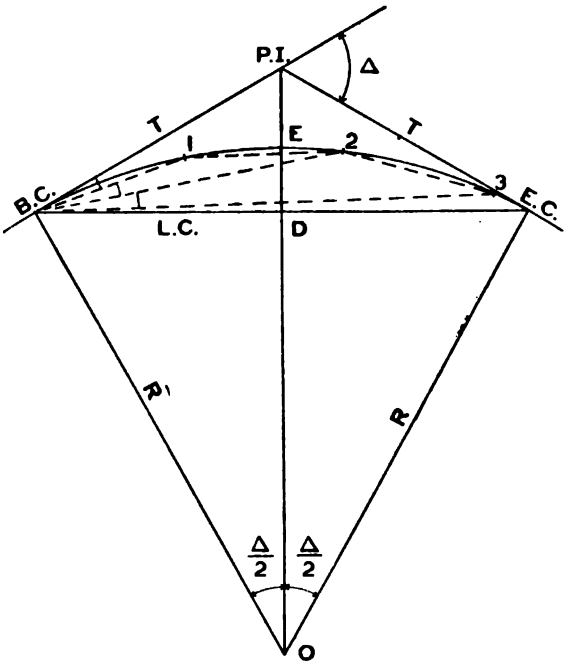

FIG. 29.

to the radii (R, R) forming the former. Also the straight line joining O (the centre of the circle) with P.I. bisects the central angle making each half of it equal to $\frac{1}{2}\Delta$.

The straight line joining B.C. to E.C. is bisected by this line O P.I. in D at right angles. This line B.C. E.C. is commonly called the *long chord*, as it joins the extremities of the curve. ED the ordinate in the centre to the curve is termed the *middle ordinate*, and P.I. E is called the *external distance*.

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All these can be expressed in terms of the radius (R), and functions of the external angle $\triangle$.

$$\text{Thus the subtangent T} = \text{R tan } \tfrac{1}{2}\triangle \quad . \; . \; . \quad (c)$$
$$\text{The long chord (L.C.)} = 2\text{R sin } \tfrac{1}{2}\triangle \quad . \; . \; . \quad (d)$$
$$\text{The middle ordinate (E.D.)} = \text{R vers } \tfrac{1}{2}\triangle \quad . \; . \; . \quad (e)$$
$$\text{The external distance} = \text{R exsec } \tfrac{1}{2}\triangle \quad . \; . \; . \quad (f)$$

(the versine = 1 − cosine and the exsecant = secant − 1)

The chain points measured from the B.C. are shown at 1, 2 and 3 in Fig. 29, the total length of curve here being about 3·1 chains. Therefore, if the chain is 100 feet long, the total length of curve = 310 feet. Thus distances B.C. 1, 1 2 and 2 3 subtend respectively at the centre of the circle angles each of the value D° (the degree of the curve).

For example, if the curve is a 5° one, there will be three of these 5° angles subtended by the arc B.C. 3, and ·1 of one 5° angle (or 30′) subtended by 3 E.C. Therefore the whole central angle or $\triangle$ will be = 15° 30′.

If B.C. 1, B.C. 2 and B.C. 3, be joined respectively by straight lines, the angles P.I. B.C. 1, 1 B.C. 2 and 2 B.C. 3, are each equal. This is a property of circles (see following paragraph in parenthesis).

(Properties of Circles used in Setting them out on the Ground by Deflection Angles.—In Fig. 30 AOD is a diameter of circle. AE a tangent at its extremity, and therefore at right angles to it.

AB and BC are equal arcs. Join AB, AC, OB, and OC. Bisect AB in F and join OF.

Now angle FAO + angle AOF = 90° = angle EAO.

$$\therefore \text{angle EAF} = \text{angle AOF} = \tfrac{1}{2} \text{ angle AOB.}$$

Similarly angle EAC = $\frac{1}{2}$ angle AOC.

$$\therefore \text{angle BAC} = \tfrac{1}{2} \text{ angle BOC.}$$

Thus, if a system of equal arcs AB, BC, etc., be set off round the circumference of a circle, and a tangent be drawn at the first point (A), and chords be drawn from the first point to each of the other points, a splay of radiating lines, cutting off equal angles, is constructed. Each of these angles, namely that between the tangent and first chord, that between the first and second chords, and so on, is equal to half the angle at the centre of the circle, subtended by any one of the equal arcs.

In setting out a railway curve these equal arcs, or their corresponding chords, are chain lengths, and the angles they subtend at the centre of the circle are equal to D, the degree of the curve. Also the angles in the splay at A are each equal to $\frac{1}{2}$D. Therefore the deflection angles turned in setting out a circular curve from a point on the circumference (as A) are each equal to half the degree of the curve when the arcs are each a chain length. It is on account of this property that a circular curve can be set out so simply by a transit theodolite from any point on its circumference, without the need of finding the centre of curvature, which in the case of railway curves is too far distant. But

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in the case of street railways, or tram lines, curves can usually be struck from a centre, as the curvature is bound to be sharp in turning street corners.)

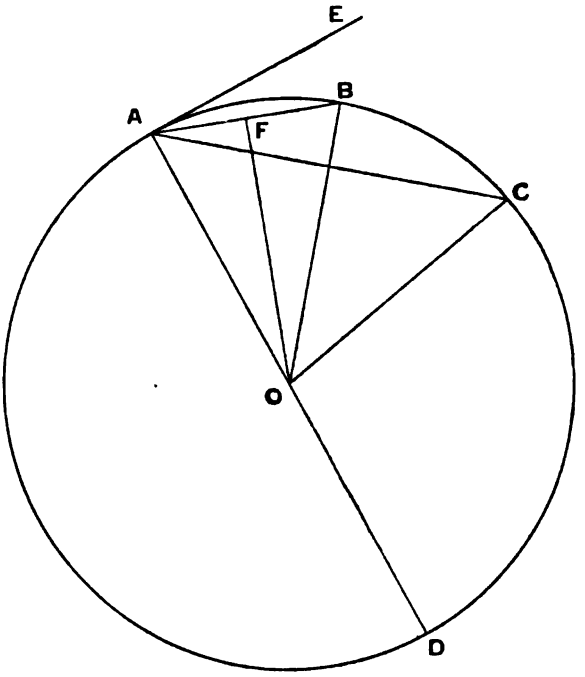

FIG. 30.

Deflection Angle Method of Setting Out Railway Curves.—When setting out the circular curve by deflection angles from the B.C. it is only necessary, after setting the angle at zero, or 360°, on the instrument, to sight on the P.I., and clamp the main limb or bottom plate of the instrument. Then unclamping the vernier plate, set an angle of $\frac{1}{2}$D°, and starting the chainmen chaining from the B.C., make them locate point 1 one chain length from B.C., and on the line of sight of the instrument. Point 1 having been accurately fixed, a further $\frac{1}{2}$D° is set on the instrument, and the fore chainman pulling the chain forward another chain length, the rear man, remaining at point 1, endeavours to swing the line of chain round till his extremity is on the line of sight of the instrument. A skilled chainman will approximate to the correct point almost at the first trial, and only needs to alter his arrow a few inches when lined-in by the instrument. After this he makes a further adjustment by tightening up his chain, and having a final "check for line" made by the instrument. In a similar manner the curve is set out for any number of chain lengths according to its length.

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The Chord Offset Method of Finding Points on the Curve.—It is advisable for a chainman to get accustomed to measuring the chord offsets as X2, Y3, and Z4 in Fig. 31. These, if measured at chain points, will all be equal. Accordingly, if when point 1 is set out the fore chainman pulls out his chain to X, in line with 0 and 1, and marks the point X, and then swings his chain into the line 1 2 when directed by the instrument man, and fixes point 2, he can measure X2 accurately with his 2-foot rule, or a ranging pole, and when the chain is pulled out to Y set off the distance X2, and fix the point 3 almost correctly, only needing final correction by the instrument. In the same way all subsequent points can be set out. In this manner it is possible to set out a curve almost as accurately as when using a

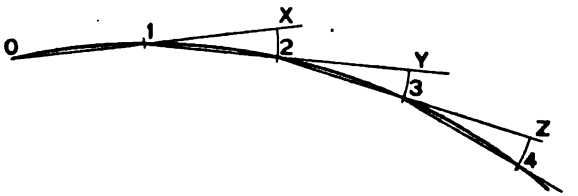

FIG. 31.

theodolite, but in order to do so the chord offset should be calculated instead of measuring it roughly on the ground.

In making use of chord offsets to set out a curve, care must be taken that the first measured offset is taken when lining in on two points 1 chain apart. Thus if 0 1 was not a chain length but less, which frequently happens in the case of the tangent point and the next chain point on the curve, due to the tangent point not occurring at the even chainage, the chord offset would be less than the correct length, and consequently if used to "set out" the rest of the curve, it will cause it to be too flat.

Determining the Deflection Angle for Fractions of a Chain Length.—Now that points 1, 2 and 3 are fixed (Fig. 29), it is necessary to set off the angle for the last point, the E.C. The distance from 3 to E.C. is ·1 chain, consequently the deflection angle is $\cdot 1 \times \frac{1}{2}D°$, *i.e.* $\frac{D''}{20}$, which in the case of a 5″ curve is 15′, and the total angle to be set out from the tangent will be $3 \times \frac{5°}{2} + 15' = 7° 45'$. If this angle = $\frac{1}{2}\triangle$ the summing of the deflection angles has been correct, and the point E.C. established in this way should coincide with the E.C. previously fixed; if not the operation has not been carried out with sufficient care.

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Setting out Railway Curves

When the transitman starts "setting out" curves on the ground he finds there only the tangents stationed throughout, and their intersection points staked out. These latter are the P.I.s. The first operation is to set up the transit at the P.I. and measure the intersection angle ($\triangle$). An infinite number of circular curves can be fitted to touch these tangents, provided they are long enough, and the flatter each curve is the greater is its subtangent.

It is next necessary to calculate the length of the subtangent in order to fix the B.C. and E.C., each of which is carefully found by measuring the length of the subtangent from the P.I. along the tangents.

Having decided upon the degree of the curve, we find the subtangent as follows :—

$$T = R \tan \tfrac{1}{2}\triangle \quad \text{from } (c)$$

$$\text{But } R = \frac{50}{\sin \frac{1}{2}D} \quad \text{see } (a)$$

$$\therefore T = 50 \cdot \frac{\tan \frac{1}{2}\triangle}{\sin \frac{1}{2}D} \quad \text{here both } \triangle \text{ and D are known.}$$

It now becomes necessary to find the chainage of the B.C. The chainage of the P.I. is known, therefore the chainage of the B.C. is found by deducting the length of the subtangent from it. As the new chainage will be carried round the curve (not round the subtangents) the new chainage of the E.C. will be less than before. Whatever chainage may be found for the B.C., the next even chain point and the subsequent ones are set out round the curve. This generally means that between the B.C. and the first chain point on the curve, and between the E.C. and the last chain point are irregular lengths less than a chain. It is assumed, in calculating the deflection angles for these irregular lengths, that the angles bear the same ratio to $\frac{D}{2}$ which the length bears to the whole chain length. This, though only approximate, is sufficiently correct for general field work, but would not be so in setting out the centre line of a tunnel, or any special work.

Practical Example.—The following is a practical example which will make these explanations quite clear.

Two tangents intersecting at P.I., Fig. 32. It is decided to connect them by a 3° curve. The transit is set up at P.I. and the external intersection measured. This is found to be 17° 40′.

It is now necessary to calculate the length of the subtangent T.

$$T = 50 \times \frac{\tan \frac{1}{2}(17° 40')}{\sin \frac{1}{2}(3°)} = 50 \times \frac{\tan 8° 50'}{\sin 1° 30'} = 50 \times \frac{\cdot 15540}{\cdot 02618} = 296 \cdot 8 \text{ feet.}$$

see (c) and (a)

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Now the chainage of the P.I. is 70 + 36 which length, less 296·8, gives the chainage of the B.C. as 67 + 39·2. It is next necessary to calculate the length of the curve. For every 3° in the central angle there will be 100 feet on the curve (if 100-foot chain is being used). But the central angle = 17° 40′

$$\therefore \text{ length of curve} = \frac{17\frac{2}{3}^\circ}{3^\circ} \times 100 = 588{\cdot}9 \text{ feet.}$$

∴ The chainage of the E.C. = (67 + 39·2) + 588·9 = 73 + 28·1

(The plus sign in thus marking chainage is used in place of a decimal point to make the marking more distinct. Thus 73 + 28·1 means 73·281 chains, it also means in this case 7328·1 feet.)

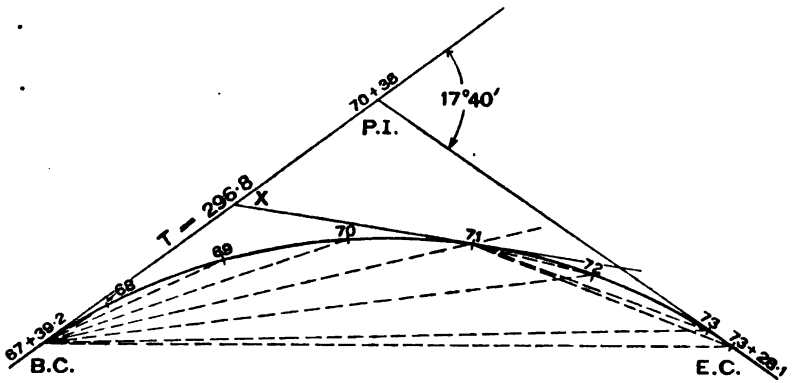

FIG. 32.

It is interesting to note how much less is the chainage of the E.C. when the measurements from the B.C. to the E.C. are taken along the curve instead of on the tangents. The length along the tangents = 2 × 296·8 = 593·6, whilst the length of the curve (from above) = 588·9. Thus giving 593·6 − 588·9 = 4·7 feet, which would have made the chainage of the E.C. 73 + 32·8.

The actual position of the E.C. on the ground is marked out at the same time as the B.C. by chaining from the P.I. 296·8 along the second tangent.

It is now only necessary to set up the theodolite at the B.C., and fix the positions of the chain points on the curve by deflection angles and chaining. The vernier on the instrument is set to read zero or 360° and the line of collimation is set upon the P.I., and then the bottom limb clamped. Then the vernier plate is unclamped, and the angles for the stations 68, 69, 70, etc., are set in turn and the stations fixed. Since the chainage of the B.C. is 67 + 39·2 it requires 60·8 feet to bring it up to 68 + 00.

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Now the deflection angle for one chain length (100 foot) is $\frac{D}{2} = \frac{3^\circ}{2}$ $= 1^\circ\ 30'$ or $90'$.

$\therefore$ Deflection angle for 60·8 feet $= \frac{60{\cdot}8}{100} \times 90' = 54'\ 43{\cdot}2''$.

This angle must be turned off on the instrument, and station 68 + 00 fixed by chainman measuring 60·8 feet from the B.C.

The deflection angle for the next station 69 + 00 is obtained by adding 1° 30′, the deflection angle $\left(\frac{D}{2}\right)$ for one chain, to that already obtained.

This will therefore be $54'\ 43{\cdot}2'' + 1^\circ\ 30' = 2^\circ\ 24'\ 43{\cdot}2''$. In the same way the subsequent angles can be found and finally that for the E.C. requires to have added to that for 73 + 00 the angle corresponding to 28·1 feet.

This is $\frac{28{\cdot}1}{100} \times 90' = 25'\ 17{\cdot}4''$.

All the angles are given in Table VIII.

TABLE VIII.

TABLE OF ANGLES.

	Sight along tangent	0° 0′ 0″
	,, on 68 + 00	54′ 43·2″
	,, ,, 69 + 00	2° 24′ 43·2″
	,, ,, 70 + 00	3° 54′ 43·2″
	,, ,, 71 + 00	5° 24′ 43·2″
	,, ,, 72 + 00	6° 54′ 43·2″
	,, ,, 73 + 00	8° 24′ 43·2″
(The E.C.)	,, ,, 73 + 28·1	8° 50′ 00·6″

The last angle, that for sighting on the E.C., should equal $\frac{1}{2}\triangle$ or $\frac{17^\circ 40}{2} = 8^\circ\ 50'\ 00''$.

The error ·6″ is due to the chainage of the B.C. and E.C., only being calculated to the first decimal place of a foot.

Observations on the Method Adopted.—It should be observed that the deflection angles given in this table are calculated to one decimal place of seconds. Needless to say, it is not possible to set off angles to such a degree of accuracy with the ordinary field transit or theodolite, the instrument work being done well if the angles are carefully set to the nearest 20 or 30 seconds, and estimated to the nearest 10 or 15 seconds. But it is necessary to calculate the deflection angles to seconds, in order that the total angles may be set as near their correct values as possible. Also the calculation work is checked by comparing the final angle with $\frac{1}{2}\triangle$, and these will not agree if this degree of accuracy is not observed.

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Here the curve is set out in its entirety by deflection angles from the B.C., but it is not always possible to get a clear view of all the points on the curve.

Setting Out the Curve with the Instrument at any Point on it, other than the B.C.—Let it be assumed that 72 and 73 are not visible from the B.C. Then when the curve is set out as far as 71 + 00 the instrument is removed to this station and the vernier set back to the zero or 360° mark. The telescope is then transited and is made to sight on the B.C., and then the angle for 71 + 00, 5° 24′ 43″ is set to the right. This throws the line of collimation along 71 X on Fig. 26 which is tangent to the curve at 71 + 00.

Next transit the telescope and turn off a further 1° 30′ angle (to the right) which is the correct angle for 100 foot chord. This gives the direction for station 72 + 00. The actual angle which is now set on the instrument is 5° 24′ 43·2″ + 1° 30′ 0″ = 6° 54′ 43·2″, or the angle from the table for station 72.

Thus we see that by setting up the instrument at any station point, and setting the vernier to zero, and then sighting on the B.C. whose tabular angle is zero with the telescope transited, the subsequent points on the curve can be set out by re-transiting the telescope, and setting the angles from the table for the various stations required. Special attention must be paid to the fact that the instrument, when set up at any subsequent station point, must have the vernier set to the angle in the table for the station on which it is intended to sight back. Here in the case of the B.C., we set zero or 360° because 0° 0′ 0″ is the angle in the table for the B.C. ; but if it had been intended to sight back on station 69, the angle 2° 24′ 43·2″ would have been set before sighting back.

Thus the curve can be completely set out, and the last point of it should coincide with the E.C. previously set out along the tangent. It is necessary that the chainage should be carefully taken to make these points agree, and consequently the chains must be accurately tested before and after being used, or else the same chain, under the same conditions, must be used both for fixing the B.C. and E.C., and also for measuring to each chain point on the curve.

Some transitmen when running in curves fix the middle point E of the curve (Fig. 29) and immediately test the accuracy of the work by measuring the distance from E to P.I., and comparing it with the calculated external distance [see (*f*)], but this should not be necessary, except in the case of a very long curve.

In Table IX. is given a useful form of tabulation for curve angles in the field as applied to the example just given.

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TABLE IX.

FIELD NOTES.

Station.	Deflection angle for distance.	Total deflection angle set on instrument.	Bearing.	Remarks.
E.C. ⊙ 73 + 28·1	25′ 17·4″	8° 50′ 00·6″	N. 57° 27′ E.	Bearing of tangent.
73 + 00	1° 30′ 0″	8° 24 43·2″		
72 + 00	1° 30′ 0″	6° 54′ 43·2″		
71 + 00	1° 30′ 0″	5° 24′ 43·2″		
70 + 00	1° 30′ 0″	3° 54′ 43·2″		
69 + 00	1° 30′ 0″	2° 24′ 43·2″		T = 296·8 feet.
68 + 00	54′ 43·2″	54′ 43·2″		△ = 17° 40′.
B.C. ⊙ 67 + 39·2	0° 0′ 0″	0° 0′ 0″	N. 39° 47′ E.	3° curve to the right.
67 + 00				

Method to Simplify the Setting off of Deflection Angles.—When setting out a railway curve in which all points are visible from the B.C., it is found convenient to set the vernier to read the odd number of degrees and minutes due to the uneven chainage of the B.C. on the side of zero remote from that towards which the curve deflects and then to turn the telescope to sight on the P.I. The angle for the first station point beyond the B.C. will now be 0° 0′ 0″ and all subsequent station points will be determined by a simple angle.

Example.—If a 4° curve to the right is being set out and the first chain point is 20 feet past the B.C., the deflection angle for this point is $\frac{20}{100} \times 2° = 24'$. Set 359° 36′ on the vernier instead of 0° 0′ and direct the telescope to the P.I., then the angles for the station points beyond the B.C. will be 0° 0′, 2° 0′, 4° 0′, 6° 0′, etc., which are quickly and easily set out on the vernier.

OTHER METHODS OF SETTING OUT CURVES

If it is necessary to set out a curve without using a transit, one of the following methods may be adopted :—

I. **By Tangent Offsets.**—Fig. 33 shows a curve whose B.C. is at chainage 101 + 25, the subsequent chain points round the curve are 102 + 00, marked 2 in figure, 103 marked 3, etc.

The problem is to calculate the lengths of the abscissæ along the tangent B.C. *a*, *b*, *c*, *d*, and the ordinates at these points to fix the chain points on the curve.

The chord length from the B.C. to 2 is 75 feet.

∴ the abscissa of 2 on the tangent B.C.$a = 75 \cos\left(\cdot 75\frac{D}{2}\right)$ as $\left(\frac{D}{2}\right)$ is the angle for 100-foot chord, D° being the degree of the curve.

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Similarly the ordinate $a2 = 75 \sin\left(\cdot 75 \frac{D}{2}\right)$

Again, the portion of abscissa $ab = b_1 2 = 2\ 3 \cos(b_1\ 2\ 3) = 100 \cos(b_1\ 2\ 3)$.

But the angle $b_1\ 2\ 3$ = angles $b_1\ 2L + L2J + J\ 2\ 3$.

$$= \text{angle } a\text{B.C. } 2 + a\text{B.C. } 2 + \frac{D}{2}$$

In Fig. 33, b_1 2, and c_1 3, are parallel to subtangent B.C. c, 2L is the chord B.C. 2 produced, h2J is tangent at 2, and J3K tangent at 3, and 3M is the chord 2 3 produced.

$$\therefore \text{angle } b_1 2\ 3 = \cdot 75 \frac{D}{2} + \cdot 75 \frac{D}{2} + \frac{D}{2} = \cdot 75D + \frac{D}{2}$$

Similarly it can be seen that angle $c_1\ 3\ 4 = \cdot 75D + \frac{3}{2}D$.

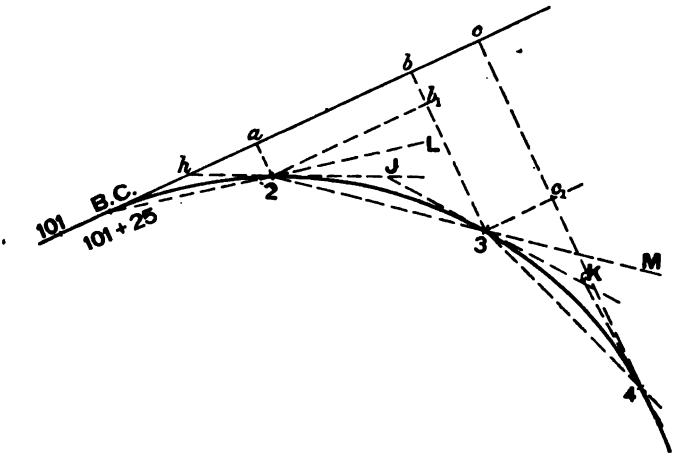

FIG. 33.

Thus the following values can be obtained for the abscissæ of 2, 3, 4, 5, etc. :—

$$\text{B.C.}a = 75 \cos\left(\cdot 75 \frac{D}{2}\right)$$

$$\text{B.C.}b = 75 \cos\left(\cdot 75 \frac{D}{2}\right) + 100 \cos\left(\cdot 75\ D + \frac{D}{2}\right)$$

$$\text{B.C.}c = 75 \cos\left(\cdot 75 \frac{D}{2}\right) + 100\left[\cos\left(\cdot 75\ D + \frac{D}{2}\right) + \cos\left(\cdot 75\ D + \tfrac{3}{2}D\right)\right]$$

$$\text{B.C.}d = 75 \cos\left(\cdot 75 \frac{D}{2}\right) + 100\left[\cos\left(\cdot 75\ D + \frac{D}{2}\right) + \cos\left(\cdot 75\ D + \tfrac{3}{2}D\right) + \cos\left(\cdot 75\ D + \tfrac{5}{2}D\right)\right]$$

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And the ordinates taking a similar form are as follows :—

$$a2 = 75 \sin\left(\cdot 75 \frac{D}{2}\right)$$

$$b3 = 75 \sin\left(\cdot 75 \frac{D}{2}\right) + 100 \sin\left(\cdot 75\ D + \frac{D}{2}\right)$$

$$c4 = 75 \sin\left(\cdot 75 \frac{D}{2}\right) + 100 \left[\sin\left(\cdot 75\ D + \frac{D}{2}\right) + \sin\left(\cdot 75\ D + \tfrac{3}{2}D\right)\right]$$

$$d5 = 75 \sin\left(\cdot 75 \frac{D}{2}\right) + 100 \left[\sin\left(\cdot 75\ D + \frac{D}{2}\right) + \sin\left(\cdot 75\ D + \tfrac{3}{2}D\right) + \sin\left(\cdot 75\ D + \tfrac{5}{2}D\right)\right]$$

These results of this example are equivalent to general expressions for tangent offsets, and can be written in a much simpler form if even chainage is taken at the B.C.

To Find the Offsets by the Method of Versines.—If it was only necessary to find the offsets, or ordinates, without finding the abscissæ, this could be done much more easily by calculating the versines for the double arcs. See Fig. 34.

Thus the first offset $a2$ = B.C.2_1 which is the middle ordinate for chord $2_2 2$.

This middle ordinate B.C.2_1 = R vers ($\cdot 75$ D) = $a2$.
Similarly „ „ B.C.3_1 = R vers ($\cdot 75$ D + D) = $b3$.
In this way all the ordinates can be obtained.

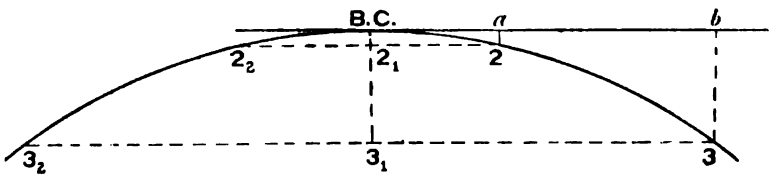

FIG. 34.

When the abscissæ and ordinates have been calculated for each point on the curve, the work of setting out the curve in the "field" is straightforward. But care is required to ensure that the offsets, especially the long ones, are accurately at right angles to the tangent at the abscissæ points.

II. **By Middle Offsets or Versines.**—The following example will show the working of this method. The first point to be found is 2 (Fig. 35). If the external offset $2h$(= R exsec.) be calculated, and set off by a square offset from a point 75 feet from B.C. along the tangent, it fixes 2 very accurately, the external offset being small ; in this case

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even for a 10° curve it is less than 15 inches. Point 2 should be checked for position by seeing that length B.C.2 is 75 feet.

Next $a2$ is calculated, it is equal to R vers (·75 D), and the double chord B.C.x = 2 R sin (·75 D).

The measuring chord is then stretched out from B.C. in a straight line, and its centre point is kept at a distance $a2$ from 2, then the point x is fixed by the extremity of the chord.

In order to fix 3 extend the chord B.C.ax to 3_1, making $a3_1 = 100$ feet, and set off offset 3_1 3 $\left(= \frac{\text{B.C.}3_1 \times 3_1x}{2\text{R}} \text{ or } \frac{175 \times 25}{2\text{R}}\right)$. Then check the position of 3 by measuring from 2 to 3, this should be 100 feet.

The rest of the curve is simply set out by calculating the chord for two chain points (*e.g.* 2 4), and the middle offset (*e.g.* $b3$). The chord

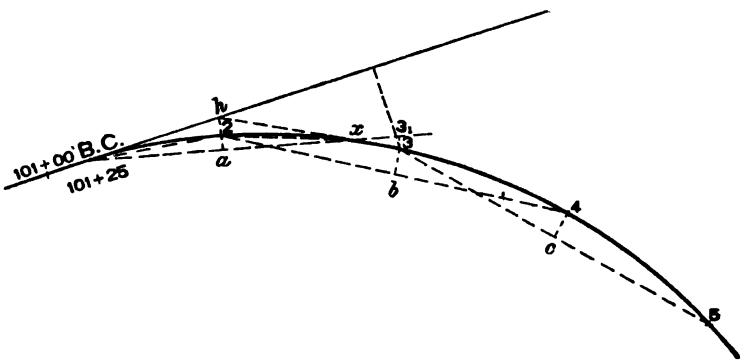

FIG. 35.

is strained from 2, and its middle point is fixed at distance $b3$ from 3, the second half of the chord being fixed accurately in line with the first half. The extremity of the chord is then at point 4.

Similarly, by straining this chord from 3, and keeping its middle point at distance $b3$ or $c4$ to the right of point 4, the next point 5 is fixed.

This method of setting out a curve must be carried out with minute accuracy, as errors are accumulative and magnified. On this account it is not recommended except for fixing one or two station points.

Problem in Railway Curves.—It sometimes becomes necessary to find the radius or degree of a curve which will be tangent to two straights, and also pass through a given point. (That is to say, a tying point frequently occurs upon a curve.)

AC and BC are tangents, and P the point through which curve must pass, PD and PE are perpendiculars on tangents (Fig. 36). P and C are fixed points ∴ PC can be measured, also angles PCA and PCE

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.. PD and PE are known, being respectively equal to PC sin PCA, and PC sin PCE.

But PD = AF, and PE = BG.

Also AF = R vers AOP = R vers $(\frac{1}{2}\Delta + \alpha)$, where α = angle COP, and BG = R vers POB = R vers $(\frac{1}{2}\Delta - \alpha)$

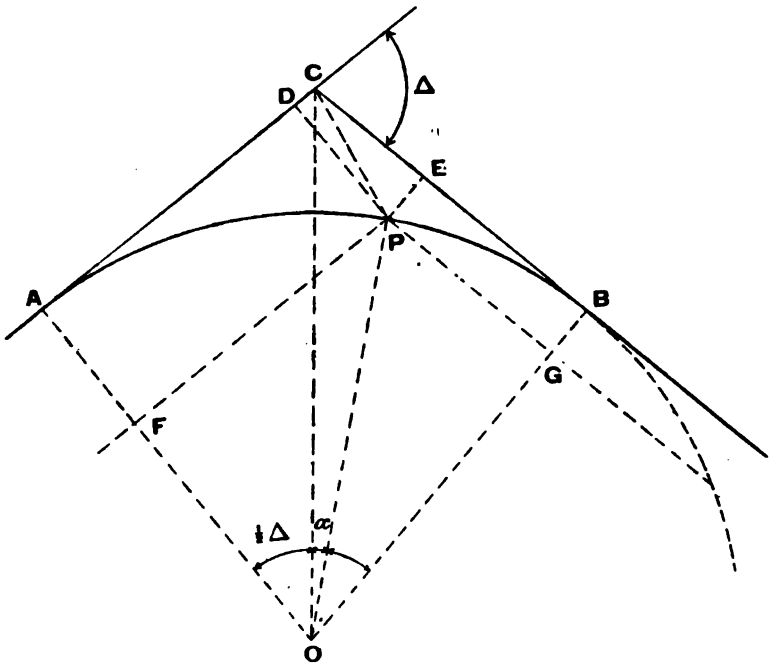

Fig. 36.

$$\therefore \frac{\text{R vers } (\frac{1}{2}\Delta + \alpha)}{\text{R vers } (\frac{1}{2}\Delta - \alpha)} = \frac{\text{PD}}{\text{PE}} = \text{K}^2 \text{ (a known ratio).}$$

$$\therefore \frac{1 - \cos(\frac{1}{2}\Delta + \alpha)}{1 - \cos(\frac{1}{2}\Delta - \alpha)} = \frac{2\sin^2\frac{1}{2}(\frac{1}{2}\Delta + \alpha)}{2\sin^2\frac{1}{2}(\frac{1}{2}\Delta - \alpha)} = \text{K}^2 \quad \therefore \frac{\sin\frac{1}{2}(\frac{1}{2}\Delta + \alpha)}{\sin\frac{1}{2}(\frac{1}{2}\Delta - \alpha)} = \text{K}$$

$$\therefore \frac{\text{K} + 1}{\text{K} - 1} = \frac{\sin\frac{1}{2}(\frac{1}{2}\Delta + \alpha) + \sin\frac{1}{2}(\frac{1}{2}\Delta - \alpha)}{\sin\frac{1}{2}(\frac{1}{2}\Delta + \alpha) - \sin\frac{1}{2}(\frac{1}{2}\Delta - \alpha)}$$

$$= \frac{2\sin\frac{1}{4}\Delta\cos\frac{1}{2}\alpha}{2\cos\frac{1}{4}\Delta\sin\frac{1}{2}\alpha}$$

$$\therefore \tan\tfrac{1}{2}\alpha = \frac{\text{K} - 1}{\text{K} + 1} \cdot \tan\tfrac{1}{4}\Delta \text{ (a known quantity)}$$

$\therefore \alpha$ has been found.

But R vers $(\frac{1}{2}\Delta + \alpha)$ = PD.

$$\therefore \text{R} = \frac{\text{PD}}{\text{vers}(\frac{1}{2}\Delta + \alpha)} \text{ (which is known).}$$

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Compound Curves

Compound curves can be used very effectively at times upon railways in any positions, but in mountainous regions their uses are often imperative.

Elements of a Compound Curve.—The following give the principal data from which most problems can be solved :—

Fig. 37 shows two arcs of circles of different radii, AB and BC,

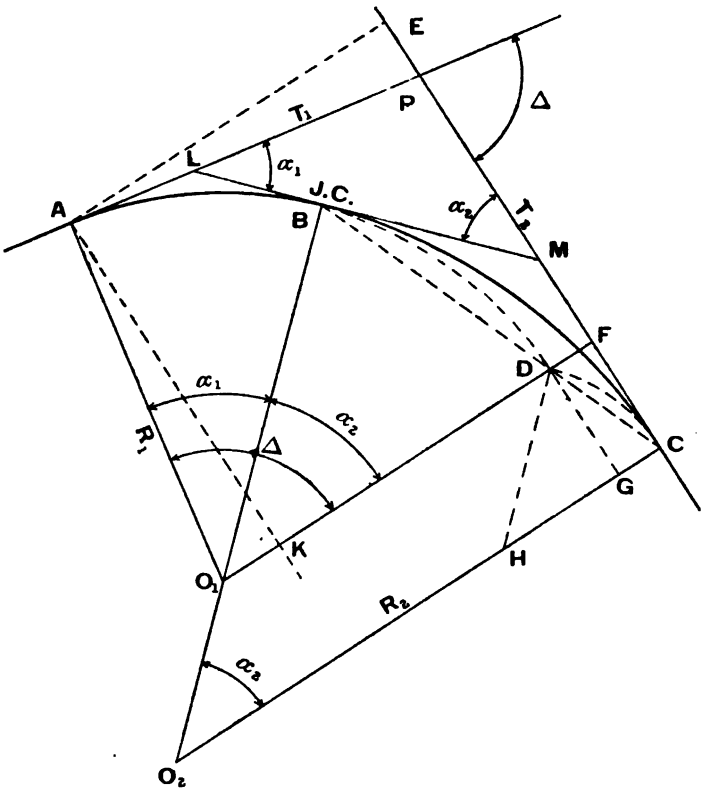

Fig. 37.

having a common tangent at B, the JC, or *junction of curves*. AB has a radius R_1 and centre at O_1, and BC a radius R_2 and centre O_2, AP (or T_1) is the tangent at A, and CP (or T_2) the tangent at C. These intersect at P with external angle Δ.

Drop perpendiculars AE and O_1F on CP, and AK on O_1F, and continue arc AB with radius R_1 to intersect O_1F in D. Drop DG perpendicular on O_2C, and draw DH parallel to BO_2.

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Draw common tangent LBM.

Join BC. This line passes through D because $\triangle$'BO_1D and BO_2C are similar.

Angle PLB = angle $AO_1B = \alpha_1$, also angle PMB = angle$BO_2C = \alpha_2$.

Now AE = KF = KD + DF = KD + GC. But AE = $T_1 \sin \triangle$, and KD = R_1 vers $\triangle$, and GC = $(R_2 - R_1)$ vers α_2, because DH = O_1O_2 = $R_2 - R_1$.

$$\therefore T_1 \sin \triangle = R_1 \text{ vers } \triangle + (R_2 - R_1) \text{ vers } \alpha_2 \quad . \quad . \quad . \quad . \quad . \quad (a)$$

It may be shown similarly that—

$$T_2 \sin \triangle = R_2 \text{ vers } \triangle - (R_2 - R_1) \text{ vers } \alpha_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad (b)$$

These two equations, together with $\alpha_1 + \alpha_2 = \triangle$ (c) should enable the solution of every ordinary soluble problem in compound curves to be arrived at.

For example, two tangents intersect externally at an angle of 20°, and it is intended to insert two curves of 6° and 4° curvature respectively, starting at a B.C. at known distance T_1 from P the intersection point. Here we require to find T_2, α_1 and α_2, the known quantities being T_1, $\triangle$, R_1 and R_2.

In eqn. (a) everything is known except α_2, which is therefore found and substituted in eqn. (c) when α_1 is found, and this being substituted in (b) T_2 is found.

Setting out the Compound Curve.—Therefore in setting out this compound curve start with the transit at the known B.C., and set out as many chain points on the 6° curve as are given by $\frac{\alpha_1^\circ}{6^\circ}$ by the ordinary method of "setting out" a simple circular curve, including any fractional part of a chain as may exist at the beginning and end of the curve to make the chainage at the chain points even. Then having fixed B (the J.C.) set up the instrument at this point, and turn the vernier to read zero on the common tangent at the J.C. Then $\frac{\alpha_2^\circ}{4^\circ}$ gives the number of chain lengths in the second part of the curve, which is now set out by deflection angles from the J.C., like any circular curve. The final point C or E.C. should agree with this point, which was fixed at the distance T_2 from the intersection point (p).

Limitations of the Curve.—It is obvious that if B.C. or T_1 is fixed, the selection of radii or degrees for the curves is not entirely optional. For instance, if T_1 is greater than or even equal to the subtangents of a simple 4° curve connecting these intersecting tangents, no compound curve of 4° and 6° (or any other degree greater than 4°) will be possible, as the compound curve started from the B.C. is too sharp to ever reach the other tangent. In the same way, if T_1 was less

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than or equal to the subtangent of a simple 6° curve used to connect these tangents, such a compound curve started at the B.C. would be bound to cut the other tangent, so no tangent point could be found. Thus it is quite evident that T_1 must have a value between that of the subtangents corresponding to the simple curves of which the compound curve is composed.

General Practical Problem.—A general type of problem which can be made use of for solving a number of problems met with in the field is the following :—

It is found necessary to alter the direction of one of the straights or tangents, which are connected by a curve of radius R_1, see Fig. 38.

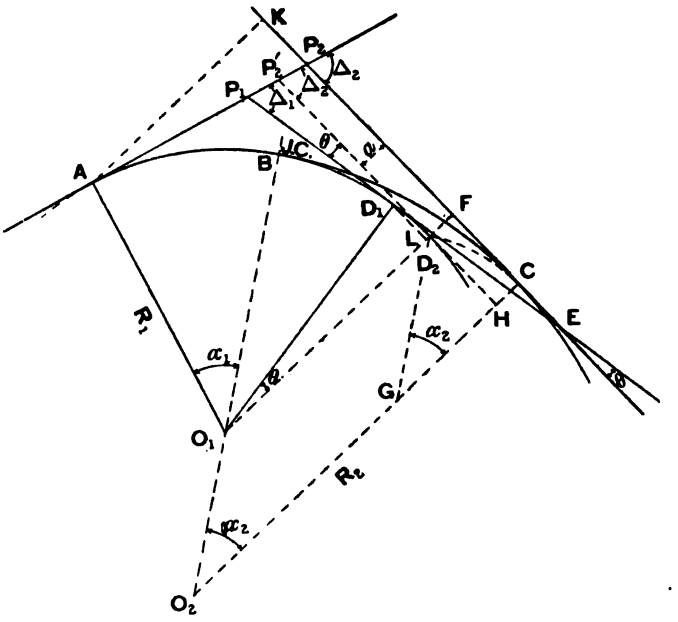

FIG. 38.

This tangent P_1D_1 is altered to the position P_2C making any angle θ, with the original tangent at point E. It is decided to preserve a portion of the existing curve, and compound it with another curve.

Now, E is the point about which the second subtangent is rotated through angle θ, which can be measured on the ground

$$\therefore \triangle_2 = \triangle_1 + \theta = \text{known.} \quad \text{Point } P_2 \text{ is also known.}$$

Drop a perpendicular from O_1 on new tangent P_2C meeting it at point F, and extend original curve ABD_1 to cut this perpendicular in D_2. Then the tangent at this point $D_2P'_2$ is parallel to CP_2.

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Let D_2G be drawn parallel to O_2O_1B, and the arc D_2C be drawn in with G as centre.

Then LF = perpendicular distance of D_1 from new tangent $P_2C = D_1E \cdot \sin\theta$, where D_1E can be measured on the ground. But $LF = LD_2 + HC = LD_2 + a = R_1 \text{ vers } \theta + (R_2 - R_1) \text{ vers } a_2$.

$$\therefore D_1E \sin\theta = R_1 \text{ vers } \theta + (R_2 - R_1) \text{ vers } a_2 \quad . \quad . \quad . \quad . \quad . \quad (d)$$

In this equation $R_2 + a_2$ are unknown. But either R_2 or the J.C. (*i.e.* point B) must be given, and the latter gives a_2. ∴ the value of R_2 can be found by (*d*).

Geometrically, if J.C. is fixed, the straight line joining it to D_2 intersects P_2E in C, the tangent point of curve R_2.

Thus each solution depends upon the selection of a suitable radius, or if the J.C. is fixed the radius R_2 can be calculated from equation (*a*).

Flattening the Ends of a Curve.—It is sometimes desirable to flatten the ends of a curve (R), as ABC in Fig. 39, by compounding it with curves of a larger radius (R_2) EF and HK. This can only be done by moving the curve in to such a position as DFGH along the line PO. Thus moving its centre to a new point O_1. Then new portions of curve will be struck from two centres O_2 and O'_2 symmetrically placed with respect to the line PO_1 produced.

When curve R_1 is moved to its new position, A moves to D and O to O_1. ∴ AD is parallel and equal to OO_1. If F be the J.C. for the new and old curves, the straight line joining FD will pass through E the tangent point or new B.C. This is evident because the △s EO_2F and DO_1F are similar. Also, if D L be drawn parallel to FO_1O_2.

$$DL = EL = R_2 - R_1$$
$$\therefore ND = EQ = (R_2 - R_1) \text{ vers } a_2$$
$$\text{But } ND = AD \cos \tfrac{1}{2}\triangle = OO_1 \cos \tfrac{1}{2}\triangle$$
$$\therefore (R_2 - R_1) \text{ vers } a_2 = OO_1 \cos \tfrac{1}{2}\triangle \quad . \quad . \quad . \quad (e)$$
$$\text{Also } a_1 + a_2 = \frac{\triangle}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (f)$$

Therefore, if it is decided what the amount of R_2 shall be, and it is immaterial how far back along the tangent the B.C. at E occurs, the position of F (the J.C.) can be placed so as to give a sufficient length FH of the original curve to ensure smooth running. Then F fixes the position of E as the straight line FD cuts the tangent in E. A further variation in F and E can be effected by altering the amount the curve is moved in OO_1 or AD. But in the ordinary course if a_1 is fixed to ensure FG being large enough, a_2 is known, see (*f*). Therefore, OO_1 is fixed by (*e*). Therefore, E the new B.C. is found because

$$AE = DQ + AN = (R_2 - R_1) \sin a_2 + OO_1 \sin \tfrac{1}{2}\triangle \quad . \quad (g)$$

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Superelevation or Cant of the Outer Rail on Curves

The centrifugal acceleration of any mass due to motion in a circular path is equal to $\frac{v^2}{gr}$ where v is its velocity in feet per second, g is the acceleration due to gravity, and r is the radius of the curve in feet.

This force, which varies as the square of the velocity, and inversely as the radius, cannot be resisted with safety by the flanges of the wheels

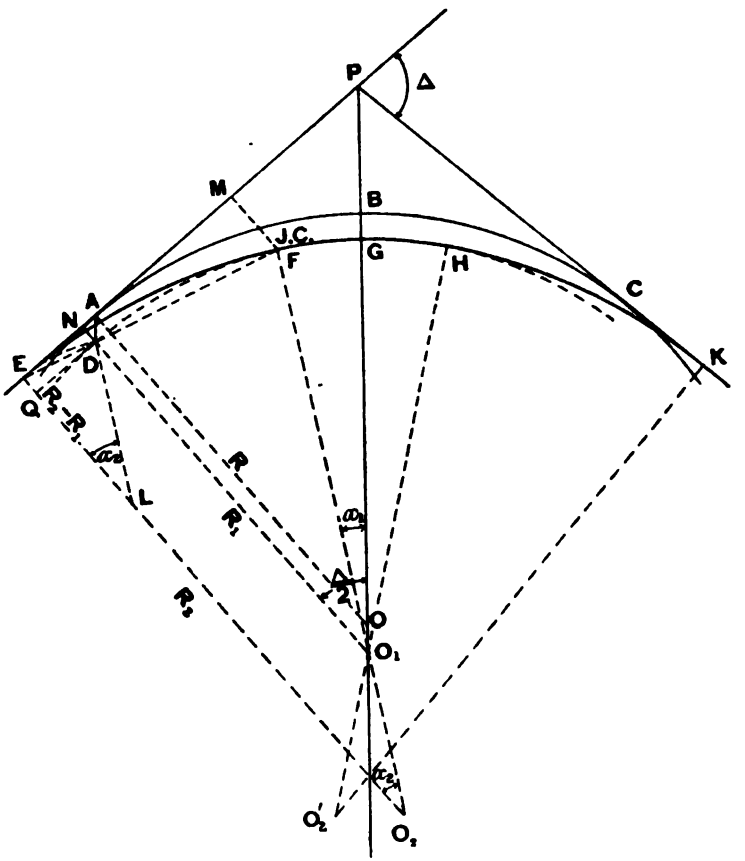

Fig. 39.

alone in railway traction, owing to the risk of derailment, but is further counteracted by canting the rails. This canting or elevating of the outer rail on a curve has the effect of throwing more of the weight upon the inner rail. The moment of this weight about a point midway between the rails will equal the moment of the centrifugal force about

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the same point (thus ensuring that the resultant of these two forces passes through this point) provided the correct superelevation has been used in laying the rails. Now the raising of one rail, say the left, above the level of the other, causes the centre of gravity of the mass moving on them to be displaced to the right through an angle equal to the angle of elevation of the outer rail above the inner one. But the tangent of this angle is the ratio of the centrifugal force to the weight

$$\therefore \text{W} \cdot \frac{v^2}{gr} \div \text{W} = \frac{\text{S}}{\text{G}}$$

where W is the weight, S the superelevation, and G the gauge or more accurately the distance between the centres of rails.

$$\therefore \text{S} = \frac{v^2}{gr} \cdot \text{G}$$

If v be replaced by V (miles per hour) *i.e* $= \frac{5280}{3600}\text{V}$, and r by R (chains radius 66-foot chain) $\therefore r = 66$ R, and g be given the value 32 and G 5·0, which is sufficiently approximate.

$$\text{Then S} = \frac{(5280)^2}{(3600)^2} \cdot \frac{\text{V}^2 \cdot 5'0}{32{\cdot}0 \times 66\text{R}} = {\cdot}0050926\,\frac{\text{V}^2}{\text{R}} \text{ feet} \quad . \quad . \quad . \quad (h)$$

$$= {\cdot}0611112\,\frac{\text{V}^2}{\text{R}} \text{ inches} \quad . \quad . \quad (h_1)$$

When the curvature is given in degrees, and the radius in 100-foot chains, using $\frac{5730}{\text{D}^\circ}$ for R, the value of S is given as follows:—

$$\text{S} = {\cdot}000704\ \text{V}^2\text{D} \text{ inches} \quad . \quad . \quad . \quad . \quad . \quad (k)$$

In the above G has been put at 5 feet; this is a little excessive, the overall measurement of the rails being 5′ 2″, and the inside measurement 4′ 8½″.

Since the superelevation varies as the square of the velocity, at high velocities it is liable to be large. Thus for a 60-mile velocity on a 30-chain curve, the superelevation is as much as 7·3″. But even as much as 10″ may be necessary in the case of high speed round a sharp curve. However, the objections to running at high speeds on sharp curves are great, and it is seldom necessary to adopt a greater cant than 6″.

Rules for Determining Superelevation.—A very useful formula for determining the cant is the following:—

$$\text{Cant in inches} = \frac{(\text{vel. in miles per hour})^2 \times (\text{gauge in feet})}{1{\cdot}25\ (\text{radius in feet})}$$

Many simple rules have been adopted for fixing the amount of superelevation. One, which has proved very useful, and has been much used

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in America, is that of elevating one inch for each degree of curvature. This is about the correct cant for a 1 degree curve when the velocity is 38 miles per hour.

A somewhat similar rule is used by the Canadian Pacific Railway for their main lines :—

(*a*) *Main lines.*—The elevation must be one inch per degree with one inch additional up to five inches, which is the maximum elevation given.

(*b*) *Branch lines.*—The elevation must be one-half inch per degree, with one inch additional up to five inches, which will be the maximum elevation used.

Thus in the case of the main line the superelevation for a 4° curve is 5 inches, which corresponds to a velocity of about 42 miles per hour.

On p. 257, particulars are given with regard to British practice in the case of superelevation. Fig. 123, p. 257, gives the usual superelevation in large black figures, and the more extreme limits, which can be adopted in special cases, in small black figures.

The Versine Method of Finding the Superelevation from the Curvature of the Rail.—Various practical methods have been adopted for determining the amount of superelevation on the ground, but the most convenient method is that which makes use of the central chord offset, or the "versine" method, as it is commonly called.

As a chord of constant length stretched between the inside of the head of the outer rail at two points, affords at its centre an offset which determines the curvature ; so inversely by varying the length of the chord according to the curvature it is possible to select it so that the middle offset shall equal the superelevation. This is possible, as the superelevation is proportional to the degree of the curve.

Now, if d be the offset at the centre of the chord, and l the length of the chord in feet—

$$d \times 2R = \left(\frac{l^2}{2}\right) \text{ where R is the radius in feet}$$

or
$$d = \frac{l^2}{8R}$$

But if R be in chains as in (*h*)—

then
$$d = \frac{1}{66} \cdot \frac{l^2}{8R}$$

But d must equal the superelevation, or S in equation (*h*)—

$$\therefore \frac{1}{66} \frac{l^2}{8R} = \cdot 0050926 \frac{V^2}{R}$$

$$\therefore l^2 = \cdot 0050926 \times 66 \times 8V^2$$

$$\therefore l^2 = 2 \cdot 68 V^2$$

$$\therefore l = 1 \cdot 64 V \quad \ldots\ldots\ldots\ldots \quad (l)$$

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Thus we see whatever the curvature may be the superelevation is determined by a chord which is directly proportional to the velocity. Thus a maximum velocity of 60 miles per hour requires the chord $l = 1{\cdot}64 \times 60 = 98{\cdot}4$ feet, whilst 40 miles per hour makes $l = 65{\cdot}6$ feet. Therefore equation (l) supplies the length of any chord whose middle ordinate, when measured, gives the correct superelevation.

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CHAPTER VIII

TRANSITION CURVES

Irregular Motion upon Approaching and Leaving a Circular Curve.—Railways have always made use of circular curves to connect their tangents or portions of straight line, in spite of the fact that the sudden change from motion in a straight to that in a circular path has long been recognized as objectionable, both from the point of view of safety, of comfort to passengers, and also of resistance to hauling. Also the superelevation, which is needed on circular curves, requires to attain its maximum value at the beginning of the curve. To make this possible it is necessary to start gradually superelevating some distance back along the straight called the "run off." The motion, due to this canting of the outer rail on the straight, is not satisfactory. The coach becomes tilted up on the side of the outer rail, the springs being depressed over the inner rail. But immediately upon passing the tangent point the centrifugal force causes the body of the coach to be thrown over to the other side, thus making much greater oscillations possible than those which could have occurred without any superelevation on the "run off." To obviate these difficulties, whilst at the same time preserving the full superelevation at all points of the circular curve, transition curves have been introduced upon all new railroads in course of construction; and even in the case of long established roads attempts are being made constantly, and in most cases successfully, to do so where bad running is known to exist.

Properties of the Transition Curve.—The property which the transition curve is required to possess is that at each point on it the superelevation shall be the correct one for the curvature at the point, assuming a fixed maximum speed for the trains. Thus a spiral, whose curvature starts from zero, and increases uniformly until it assumes the full curvature of the circular curve, will probably conform with this requirement as the superelevation will also start from zero at the beginning of the curve, and increase uniformly until it assumes the full superelevation necessary for the circular curve. On such curves, if correctly applied, the change in motion being very gradual, the

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objections or discomforts mentioned above are not experienced, Again, if superelevation was not possible on a certain portion of curve, the fact that the transition curve makes the effect of centrifugal force more gradual suggests a still more urgent need for such a curve in order to minimize as much as possible the oscillations upon entering the curve.

The discomfort[1] resulting from suddenly entering a curve from the straight is due both to the length of the wheel base, and also the radial acceleration. If b be the length of the wheel base in feet, v the velocity in feet per sec., and r the radius of curvature in feet, then the time the vehicle takes to pass any point is $\frac{b}{v}$ seconds, and the radial acceleration is $\frac{v^2}{r}$. Therefore the rate of gain of acceleration for any one vehicle is $\frac{v^2}{r} \div \frac{b}{v} = \frac{v^3}{rb}$. Thus the sudden shock, or discomfort, is proportional to the cube of the velocity and inversely to the radius, and the length of the wheel base. Consequently at high speeds, say, over 50 miles per hour, the discomfort is very much greater than that at low speeds, thus emphasizing the greater need for transition curves on flat curves, if not less than 25 or 30 chains radius (or approximately 3° 30′ or 2° 54′, with a 100-foot chord length) which are about the sharpest used for such high speeds. Now the rate of gain of acceleration is proportional to $\frac{v^3}{r}$, and this acceleration ought to be gained uniformly along the transition curve. Therefore the length of transition curve l must $= C \cdot \frac{v^3}{r}$, where C is a constant depending upon the maximum amount of oscillation to be tolerated. If C be assumed equal to unity, a value which has given very satisfactory results, $l = \frac{v^3}{r}$; or if L is the length in chains, V the velocity in miles per hour, and R the radius of the circular curve in chains, this formula becomes—

$$L = \cdot 00072428 \frac{V^3}{R} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

A simplification of this formula has been effected by assuming $11 \sqrt{R} = V$ (maximum) [a very close, practical approximation], and as the curves must be calculated for the highest velocities, substituting this value in above equation (1) reduces it to the form $L = \cdot 964 \sqrt{R}$, or if $11 \cdot 13 \sqrt{R}$ is used for the maximum velocity, a change which is on the side of safety, the equation becomes $L = \sqrt{R}$ (2), and gives a good

[1] See paper by W. H. Shortt on the "Practical method for the improvement of existing railway curves."—*Minutes of Proceedings*, Inst. C.E., vol. clxxvi., p. 99.

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practical value for the length of transition curve. This gives very efficient transition curves for circular curves of less than 56·8 chains radius, corresponding to 84 miles per hour, about the maximum used on English railways. For greater radii the length of transition curve obtained is unnecessarily long, and is best obtained from equation (1), that is, assuming a maximum speed of 84 miles per hour.

It is necessary for easy running that the total acceleration should be gained at a uniform rate along the transition curve. This being so, if it may be assumed that the speed remains constant, the acceleration is proportional to the reciprocal of the radius $\left(\frac{1}{\rho}\right)$. Therefore $\frac{1}{\rho}$ must increase uniformly along the transition curve. Thus, if λ be the length of any portion of a transition curve, and $\frac{1}{\rho}$ the reciprocal of the radius of curvature at that point, $\frac{1}{\rho} = K\lambda$ where K is a constant, also if r be the radius of the circular curve, and l the total length of the transition curve, $\frac{1}{r} = Kl$. Therefore the transition curve must satisfy the relation $\frac{1}{\rho} = \frac{1}{rl}\lambda$.

Equation of Spiral which satisfies the Conditions.[1]—A spiral whose equation is $\lambda = m\sqrt{\phi}$ (3) satisfies this condition. Here

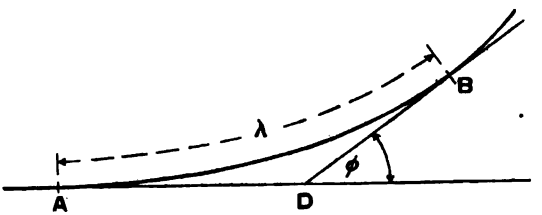

FIG. 40.

m is a constant, ϕ is the angle made with the tangent or straight AD, Fig. 40, by the tangent at the point B, B being at the distance λ from the beginning of the curve A, measured along the curve. The equation is derived as follows :—

$\frac{1}{\rho} = K\lambda$ being the condition which must be satisfied, also $\frac{d\lambda}{\rho} = d\phi$

$$\therefore \frac{d\lambda}{d\phi} = \frac{1}{K\lambda}$$

[1] "Transition Curves for Railways."—*Minutes of Proceedings*, Inst. C.E., vol. cxl., p. 161.

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Integrating this gives $\lambda = \sqrt{\frac{2\phi}{K}} + \text{constant}$ or $\lambda = m\sqrt{\phi} + \text{constant}$ where $m = \sqrt{\frac{2}{K}}$. K and therefore m being constants.

To find the value of the constant. As $\lambda = 0$ when $\phi = 0$, the constant $= 0$.

$$\therefore \text{ The equation is } \lambda = m\sqrt{\phi} \quad . \quad . \quad . \quad . \quad (3)$$

Evolution of Formulæ.—At the end of the transition if $\phi = a$ and $\lambda = l$ above equation (3) gives $l = m\sqrt{a}$.

$$\therefore m = \frac{l}{\sqrt{a}}$$

Therefore equation of curve becomes $\lambda = l\sqrt{\frac{\phi}{a}}$. . . . (4)

also $\rho = r$ when $\lambda = l$.

$$\therefore K = \frac{1}{rl}$$

$$\therefore \frac{l}{\sqrt{a}} = m = \sqrt{\frac{2}{K}} = \sqrt{2rl}$$

$\therefore$ another form of the equation is—

$$\lambda = \sqrt{2rl} \cdot \sqrt{\phi} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Next referring the curve to Cartesian co-ordinates—

$$\frac{dx}{d\lambda} = \cos\phi$$

$$\therefore dx = \cos\phi d\lambda = \frac{m\cos\phi}{2\sqrt{\phi}} \cdot d\phi$$

Expanding $\cos\phi$ gives—

$$dx = \frac{m}{2}\left(1 - \frac{\phi^2}{\underline{|2}} + \frac{\phi^4}{\underline{|4}} - \frac{\phi^6}{\underline{|6}} + \ldots\right)\frac{d\phi}{\sqrt{\phi}}$$

Integrating this gives—

$$x = m\sqrt{\phi}\left(1 - \frac{\phi^2}{5\underline{|2}} + \frac{\phi^4}{9\underline{|4}} - \frac{\phi^6}{13\underline{|6}} + \ldots\right) \quad . \quad . \quad (6)$$

Again $\frac{dy}{d\lambda} = \sin\phi$

$$\therefore dy = \sin\phi d\lambda = \frac{m\sin\phi}{2\sqrt{\phi}}\, d\phi.$$

Expanding $\sin\phi$

$$dy = \frac{m}{2}\left(\frac{\phi}{\underline{|1}} - \frac{\phi^3}{\underline{|3}} + \frac{\phi^5}{\underline{|5}} - \frac{\phi^7}{\underline{|7}} + \ldots\right)\frac{d\phi}{\sqrt{\phi}}$$

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Integrating this gives—

$$y = \frac{m\phi^{\frac{3}{2}}}{3}\left(1 - \frac{3\phi^2}{7\underline{|3}} + \frac{3\phi^4}{11\underline{|5}} - \frac{3\phi^6}{15\underline{|7}} + \cdots\right) \quad . \quad . \quad (7)$$

Assuming $\phi^{\frac{5}{2}}$ and higher powers negligible when ϕ is small equation (6) becomes—

$$x = m\sqrt{\phi} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

and equation (7)—

$$y = \frac{m\phi^{\frac{3}{2}}}{3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$\therefore \text{ eliminating } \phi \; y = \frac{m}{3}\left(\frac{x}{m}\right)^3 = \frac{x^3}{3m^2} \quad . \quad . \quad . \quad (10)$$

This is the equation of a cubic parabola.

The Cubic Parabola.—The cubic parabola has been much used for transition curves, and co-ordinate tables for different varieties of cubic parabolas have been prepared by several authors dealing with this subject. However, it must be observed that the equation of the cubic parabola was derived from (6) and (7) by only making use of the first term in the series in each. Thus it is evident that ϕ must be a small angle, if the portion of the cubic parabola needed for a transition curve is to agree closely to that given by $\lambda = m\sqrt{\phi}$. However, on railway transitions ϕ is generally small, and consequently the error is slight.

In any cubic parabola the radius of curvature decreases by a complex law until $\phi = 24° \; 5' \; 41''$, when it reaches its minimum value, after which it begins to increase. Therefore beyond this limit it is unsuitable for transition curves.

But it must also be observed that in railway spirals ϕ rarely approaches a value even half that of the limit stated, namely, $24° \; 5' \; 41''$. Even in the extreme case of a 5-chain curve, where the degree of curvature is about $17° \; 26'$, the value of ϕ is only about $13° \; 5'$, *i.e.* taking the length of transition $= \sqrt{5}$ chains, or about 150 feet. However, although this simplifies the work in setting out spirals on the ground where the scale is natural, great care must be exercised in the use of the cubic parabola in the office, if distorted scales are being used in order to magnify the changes in curvature.

The Cubic Transition Spiral.—But a curve which approximates more closely to that afforded by $\lambda = m\sqrt{\phi}$, has its equation derived by eliminating ϕ between (3) and (9).

Thus

$$y = \frac{m}{3}\phi^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$\lambda = m\sqrt{\phi} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

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$$\therefore y = \frac{m}{3} \cdot \frac{\lambda^3}{m^3} = \frac{\lambda^3}{3m^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

In the derivation of this equation the approximation of neglecting $\phi^{\frac{7}{2}}$ occurred once, and that of neglecting $\phi^{\frac{5}{2}}$ not at all.

But $$m = \frac{l}{\sqrt{a}} \text{ and } a = \frac{l}{2r}$$

$$\therefore m = \frac{l\sqrt{2r}}{\sqrt{l}} = \sqrt{2rl}$$

$$\therefore 3m^2 = 6rl$$

$$\therefore y = \frac{\lambda^3}{6rl} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

is the equation of a curve very similar to the cubic parabola, but following the curve of the original equation ($\lambda = m\sqrt{\phi}$) much more closely. This curve, which shall be called the "Cubic Transition Spiral," has its offset at any point along the curve varying as the cube of the length of the curve up to that point.[1]

Properties of Transition Spiral

In Fig. 41 ACB is the transition spiral, BD the portion of the circular curve replaced, a the value of ϕ for the end point of the curve and equal to the central angle of the circular curve subtended by the arc DB.

From equation (5) $\lambda = \sqrt{2rl}\sqrt{\phi}$

$$\therefore \text{ACB} = l = \sqrt{2rl}\sqrt{a} = \sqrt{2lra}$$

squaring gives $l^2 = 2lra \quad \therefore l = 2ra$

But $ra = \text{BD} \quad \therefore l = 2\text{BD} \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$

That is the transition curve is twice the length of the circular arc BD, C being the middle point of the curve (this is a very close approx.). Therefore for point C putting

$$\lambda = \frac{l}{2} \text{ in equation (13), gives } y = \frac{l^3}{48rl} = \frac{l^2}{48r} = \text{SC} \quad (15)$$

$$\text{But DF} = \frac{\text{BD}^2}{2r} = \frac{l^2}{8r}, \text{ and at B, } y = \frac{l^3}{6rl} = \frac{l^2}{6r}$$

$$\therefore \text{SD} = \text{BE} - \text{DF} = \frac{l^2}{6r} - \frac{l^2}{8r} = \frac{l^2}{24r}^* \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

* Here BD is taken equal to the arc instead of the chord.

[1] The complete equation giving y in terms of λ may be derived from (7) by substituting $\frac{\lambda^2}{m^2}$ for ϕ, and is

$$y = \frac{\lambda^3}{8m^3}\left(1 - \frac{3\lambda^4}{7\ \underline{|3}\ m^4} + \frac{3\lambda^8}{11\ \underline{|5}\ m^8} - \frac{3\lambda^{12}}{15\ \underline{|7}\ m^{12}} + \cdots\right) \quad . \quad . \quad (12)$$

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SD, which is called the "shift," is designated by the letter S, and is the minimum distance between the circular curve and the tangent or

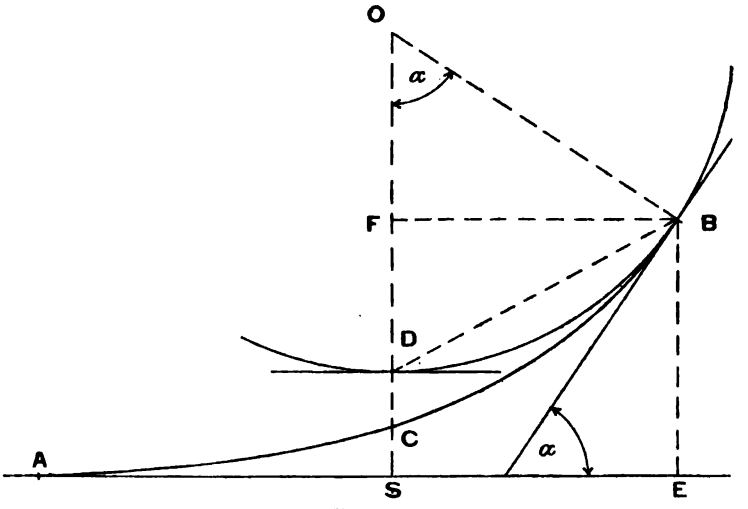

FIG. 41.

straight. [*Note.*—In using formula $l = \sqrt{r}$ as suggested on p. 124, S becomes constant and equal to $\frac{1}{24}$ chains or 2′ 9″.]

But SC was shown to be equal to $\frac{l^2}{48r}$. Therefore SD is bisected by the spiral at C, and C is also the central point of the spiral.

Again $$BE = \frac{l^2}{6r} = 4 \cdot \frac{l^2}{24r} = 4S \quad . \quad . \quad . \quad . \quad . \quad (17)$$

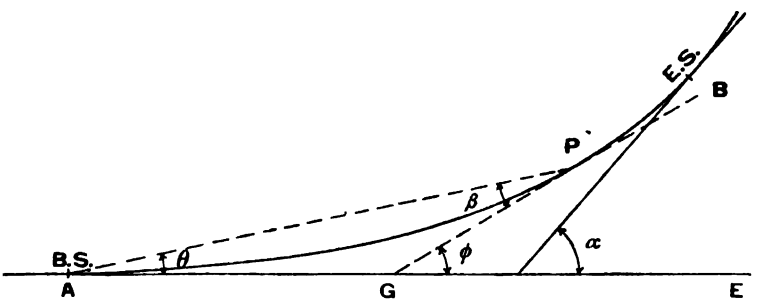

FIG. 42.

Now take any point P on the transition curve and join it to A by chord PA (Fig. 42), and let the angle PAE be θ.

$$\tan \theta = \frac{y}{x}$$

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but the equation of the curve is of the form—

$$y = nx^3 \quad \therefore \frac{dy}{dx} = 3nx^2 = \tan\phi$$

$$\therefore \tan\phi = 3nx^2 = \frac{3y}{x} = 3\tan\theta$$

$$\therefore \text{for small values of } \phi,\ \theta = \tfrac{1}{3}\phi$$

$$\therefore \text{angle APG or } \beta = \tfrac{2}{3}\phi = 2\theta.$$

The error due to making this assumption is imperceptible in all ordinary cases. For a 9-chain curve the spiral angle θ, for the last point on the transition, will have an error of 9 seconds, the error being less for other points on the curve. In the case of a 5-chain curve, which is one of extremely sharp curvature, the error at the end point of a transition curve $2\frac{1}{2}$ chains long is 22 seconds.

The true value of θ derived from the previous equations is given by—

$$\tan\theta = \frac{\phi}{3} + \frac{\phi^3}{105} + \frac{\phi^5}{5997} + \frac{\phi^7}{198700} + \cdots$$

Again, by (13) substituting l_1 for λ at any point along the curve and letting the deflection angle at A be θ_1 equation (13) becomes—

$$y = \frac{{l_1}^3}{6rl}$$

But for small values of ϕ—

$$\theta_1 = \sin\theta_1 = \frac{y}{l_1}$$

$$\therefore \theta_1 = \frac{{l_1}^3}{6rl} \cdot \frac{1}{l_1} = \frac{{l_1}^2}{6rl} \quad \ldots\ldots\ldots \quad (18)$$

Similarly for a second point—

$$\theta_2 = \frac{{l_2}^2}{6rl}$$

$$\therefore \frac{\theta_1}{\theta_2} = \frac{{l_1}^2}{{l_2}^2} \quad \ldots\ldots\ldots \quad (19)$$

Thus, if θ_n for the end point (B) of the transition is known, θ_λ for any other point (λ) on the curve is given by $\theta_\lambda = \frac{\lambda^2}{l^2}\theta_n$ where the factor $\frac{\lambda}{l}$ is the fraction, λ is of the whole length of the transition curve.

Setting out the Transition Spiral by Deflection Angles

From above properties in setting out transition spirals by deflection angles from the B.S. (beginning of spiral) the deflection angles vary as the squares of the lengths measured along the curve from the B.S.

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Also for the E.S. (end of the spiral), where it joins the circular curve—

$$\theta = \frac{l^2}{6rl} = \frac{l}{6r}$$

$$\therefore \alpha = 3\theta = \frac{l}{2r} \text{ at point E.S.}$$

Therefore if it is decided to set out the spiral by dividing its total length l into any number of equal parts say eight, and if the instrument is set up at A, the eight deflection angles measured from the tangent are found as follows:—

Since $$\alpha = \frac{l}{2r}, \theta_8 = \tfrac{1}{3}\alpha = \frac{l}{6r}$$

$$\therefore \theta_1 = \theta_8(\tfrac{1}{8})^2$$
$$\theta_2 = \theta_8(\tfrac{2}{8}) = \theta_8(\tfrac{1}{4})^2$$
$$\theta_3 = \theta_8(\tfrac{3}{8})^2$$
$$\theta_4 = \theta_8(\tfrac{1}{2})^2$$

Thus the operation of calculating the deflection angle θ for any point on the curve can be easily and rapidly performed in the field without the need of any offset tables, as $\theta_n = \frac{l^2}{6rl} = \frac{l}{6r}$, which can be determined immediately the length of the transition is decided upon. Consequently, the operation of setting out the cubic transition spiral by deflection angles from the spiral tangent point is as simple as setting out the ordinary circular curve by deflection angles.

Example.—If a transition spiral is required to connect a straight with a 25-chain curve, the degree of this curve for 100-foot chord length is 3° 28′ 20″.

Taking the length of transition = $\sqrt{R}$, gives l = 5 chains.

Now $$\alpha = \frac{l}{2r} = \tfrac{5}{50} = \cdot 1$$

$$\therefore \alpha^\circ = \cdot 1 \times 57{\cdot}3^\circ = 5{\cdot}73^\circ = 5^\circ\ 43'\ 48''$$
$$\therefore \theta \text{ (for the E.S.)} = \tfrac{1}{3}\alpha = 1^\circ\ 54'\ 36''$$

If the points on the spiral are put in at every half-chain there will be 10 deflection angles as follows:—

$$\theta_1 = (1^\circ\ 54'\ 36'')(\tfrac{1}{10})^2 = 1'\ 8{\cdot}76''$$
$$\theta_2 = (1^\circ\ 54'\ 36'')(\tfrac{1}{5})^2 = 4'\ 35{\cdot}0''$$
$$\theta_3 = (1^\circ\ 54'\ 36'')(\tfrac{3}{10})^2 = 10'\ 18{\cdot}84''$$
$$\theta_4 = (1^\circ\ 54'\ 36'')(\tfrac{2}{5})^2 = 18'\ 20{\cdot}16''$$

similarly, the remaining angles are 28′ 39″, 41′ 15·36″, 56′ 9·24″, 1° 13′ 20·64″, 1° 32′ 49·56″, and 1° 54′ 36″.

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Setting out the Spiral by Deflection Angles from the "End of Spiral."—It is frequently found convenient to set out a portion or all of the transition curve by deflection angles from the E.S. (end of spiral), point B on Fig. 41, and it may be observed (see p. 144) that the spiral curve deviates from the circular curve at the same rate per length of spiral, measured from B, as it does from the tangent AE per length of spiral measured from A. Therefore, the set of deflection angles prepared for setting out the spiral from point A, may be used for setting out from point B, by deducting each in turn from the corresponding angle for the circular curve. Thus in the previous example, in the case of a 25-chain curve, the deflection angles for half-chain points on the circular curve are as follows :—

$$\frac{1718{\cdot}87}{R} = \frac{1718{\cdot}87}{25} = 68' 45''$$ deflection angle for full chain chord

$\therefore$ 34′ 22·5″ = deflection angle for half-chain chord.

Therefore the deflection angles to the 10 points on the spiral are given by Table X.

TABLE X.

	1. Deflection angles measured from tangent at B to points on the circular curve every 33 feet.	2. Corresponding values of θ, etc., found on p. 131.	3. Difference between 1 and 2 columns giving deflections angles to spiral.
1	34′ 22·5″	1′ 8·76″	33′ 13·74″
2	1° 8′ 45″	4′ 35·0″	1° 4′ 10·00″
3	1° 43′ 7·5″	10′ 18·84″	1° 32′ 48·86″
4	2° 17′ 30″	18′ 20·16″	1° 59′ 9·84″
5	2° 51′ 52·5″	28′ 39″	2° 23′ 13·50″
6	3° 26′ 15″	41′ 15·36″	2° 44′ 59·64″
7	4° 00′ 37·5″	56′ 9·24″	3° 4′ 28·26″
8	4° 35′ 00″	1° 13′ 20·64″	3° 21′ 39·36″
9	5° 9′ 22·5″	1° 32′ 49·56″	3° 36′ 32·94″
10	5° 43′ 45″	1° 54′ 36″	3° 49′ 9·00″

It may be observed that the last angle 3° 49′ 9″, which is the angle which must be deflected from the tangent at B in order to sight on A, when divided by 2 (*i.e.* 1° 54′ 34·5″) is equal to the angle deflected from the tangent at A in order to sight on B. The slight discrepancy of 1·5″ between 34·5″ and 36″, which would not be perceptible in the field, is due to the approximate method of finding the value of α on p. 131.

Setting out Spiral by Deflection Angles from any Intermediate Point.—In the event of its being impossible to set out the whole transition curve with the transit set at the B.S. or at the E.S., it

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will become necessary to set the instrument at some intermediate point on the spiral, when the problem of finding the deflection angles will arise. In the transition spiral the degree of curvature increases uniformly with the distance measured along the curve. Therefore the degree of curvature at any chord point along the curve is easily calculated by dividing the number of chords up to the point by the total number of chords in the spiral and multiplying by the degree of the circular curve.

In Fig. 43 KGL is the circular curve of the same degree of curvature as the portion of the spiral at G, HG being the common

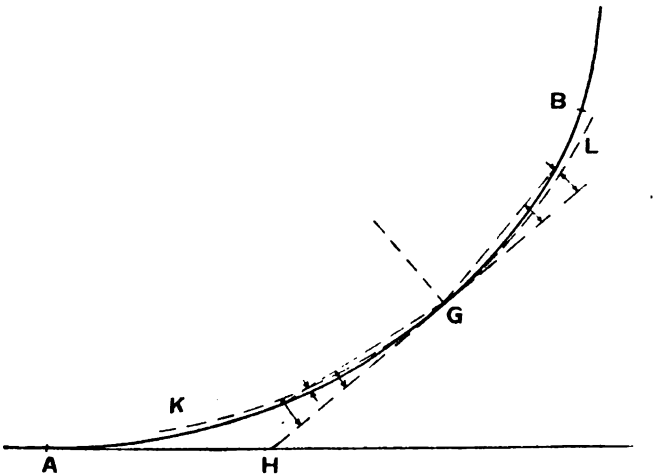

FIG. 43.

tangent to both curves. But the spiral will deviate from the circular curve at the same rate per length measured from G as it does from the tangent AH per length measured from A (see p. 144). Therefore the angle which must be deflected from the tangent by the instrument at G in order to locate any point beyond G is the sum of the deflection angle to the corresponding point on the circular curve KGL, and the spiral deflection angle at A for the same chord length measured from A. Again, in locating points between G and A the difference instead of the sum of these deflection angles is taken.

Example.—Thus in the previous example if it is decided to set the instrument at point 7 the degree of curvature at this point

$$= \tfrac{7}{10} \times \text{degree of curvature of 25-chain curve}$$
$$= \tfrac{7}{10} \times (2^\circ\ 17'\ 30'')^* = 1^\circ\ 36'\ 15''$$

* Note! The degree of curvature here is assumed to be the central angle subtended by a 66-foot chord.

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Therefore the deflection angle for ½-chain length = ¼ (1° 36′ 15″) or 24′ 4″ approximately.

Therefore the deflection angles required to fix points 8, 9, 10, 6, 5 and 4 are found in Table XI.

TABLE XI.

1. Instrument at	2. To sight on.	3. Angle between tangent and point on circular curve.	4. Deflection angles at A to points on spiral for corresponding distances.	5. Required angle algebraic sum of 3 and 4.
7	Along tangent at 7	0° 0′ 0″	0° 0′ 0″	0° 0′ 0″
7	6	24′ 4″	1′ 9″	22′ 55″
7	5	48′ 8″	4′ 35″	43′ 33″
7	4	1° 12′ 12″	10′ 19″	1° 1′ 53″
7	Sighting opposite way along tangent	0° 0′ 0″	0° 0′ 0″	0° 0′ 0″
7	8	24′ 4″	1′ 9″	25′ 13″
7	9	48′ 8″	4′ 35″	52′ 43″
7	10	1° 12′ 12″	10′ 19″	1° 22′ 31″

In order to set the instrument to read zero angle on the tangent at the point (7), *i.e.* along GH in figure, it must be remembered that the angle AGH = twice the angle GAH.

Therefore on the instrument set up at G set the vernier to read twice the angle GAH and transiting the telescope make it sight upon point A, then clamp the lower limb and make a fine adjustment ; then unclamping the vernier plate set it to read 0° 0′ 0″ when the telescope will sight along the tangent at G and the deflection angles to the spiral can be set off directly.

Example where Curvature is given in Degrees instead of Chains Radius.—It must be observed that whether the circular curve is given in degrees or chains radius the problem of connecting it and the tangent by a suitable transition curve is the same.

Thus, for example, to connect a 4° curve with a tangent, if the length of transition is not already fixed, it may be found as before by taking for it the value $\sqrt{R}$ or $\sqrt{\left(\frac{50}{\sin 2^\circ} \times \frac{1}{66}\right)} = \sqrt{\left(\frac{50}{\cdot 0349 \times 66}\right)}$ which gives $l = 4{\cdot}66$ chains, also $R = 21{\cdot}7$ chains.

The rest of the operation is as previously described. In this case the spiral might be set out by taking seven equal chain lengths of ⅔ chain or 44 feet, but in that case the centre point of spiral (C) would not be fixed on the ground. If the point C is required it will

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occur at the end of the seventh chord where the chord lengths are taken as 22 feet. (If found more convenient any other number of chord lengths, as 8 or 10, for fixing the 4·66 chains may be used.)

Thus, again, it is unnecessary to be provided with special tables of angles in order to set out the transition spiral by deflection angles, as the operation of calculating the angles is a simple matter. In the case of locating a new road it is preferable generally to set out the spirals by deflection angles; on the other hand, where existing roads require altering into a spiral curve, offsets are more convenient but are generally measured from the existing rail. This work will be dealt with later.

The Case where the Shift is Constant.—Where the length of the spiral is determined by extracting the square root of the radius of curvature of the circular curve, it may be noticed that the shift becomes constant.

Thus taking $l = \sqrt{r}$ (see p. 124), and as S (the shift) $= \frac{l^2}{24r}$ (see (16)), substituting in the second equation the value of l from the first we have $S = \frac{r}{24r} = \frac{1}{24}$ of a chain.

This value $= \frac{1}{24} \times 66 = 2'\ 9''$, and will be constant for all cases where l is assumed equal to $\sqrt{r}$.

However, the assumption that $l = \sqrt{r}$ is neither economical nor suitable for curves of higher radius than 54·3 chains. In that case equation (1) (p. 124) must be used to determine L.

Assuming the maximum velocity to be 82 miles per hour this formula becomes $L = \frac{399 \cdot 19}{R}$ which determines the value of L, which when substituted in equation (16) (p. 128) gives the value of S.

Setting out Curves by Offsets

In all new work it is simpler and more satisfactory to set out transition curves by deflection angles if the curve lends itself to that method. But in the case of existing lines, which it is intended to improve, especially where there is much traffic, it is found more convenient to set out transition curves by offsets. For the cubic parabola these are to be found in tables for a large variety of curves, giving the lengths along the tangent and the corresponding square offsets, and as the cubic parabola approximates very closely to the cubic transition spiral for small values of ϕ, these tables may be used in most cases. But it is not essential to use the cubic parabola for this purpose, as the equation $\left(y = \frac{\lambda^3}{6rl}\right)$ of the transition spiral will give the offsets

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in relation to definite lengths measured along the curve, with which data the curve is easily run in on the ground by "setting out" with a wooden square and sliding it along the tangent line with the required offset marked on it by an arrow or pin sticking in it. By sliding this until the chainage agrees with the arrow, the required point can be found on the ground and accurately fixed by testing the measurements a second time before moving on to the next point. If it should be desired to fix pegs along the tangent for future testing of the road for alignment by offset measurements, these points can be fixed at the same time as the points along the transition curve. If it is decided to do the work of setting out with offsets by using the abscissæ along the tangent, these can be determined for each point by $x = \sqrt{(\lambda^2 - y^2)}$ and a table of them prepared in the office before going on the ground.

For offset work the values of y are most easily determined from the shift $\left(\frac{l^2}{24r}\right)$ which is one-quarter of the end offset, and as the equation of the curve gives the offsets varying as the cube of the distance, y for any point, say $\frac{1}{8}$ point on the curve is given by $y = (\frac{1}{8})^3\ 4S$.

Note! It is only necessary to calculate the offsets of the curve beyond the shift, as the same offsets can be used for the second half of the spiral by measuring from the circular curve instead of the tangent. However, it is not always convenient to do this, as it involves fixing the end E.S. (end of spiral) and the other corresponding points on the circular curve.

There are two principal methods of doing this—

(*a*) by setting out the curve completely by tangent offsets, and

(*b*) by setting out the first half of the curve, from the P.S. to the shift by tangent offsets, and the second half by offsets from the circular curve.

Case A.—Since equation (10) shows y to be proportional to x^3, we only need to find the value of BE equation (17) or the end offset of the curve which is equal to 4S, and therefore known, and all the offsets to the curve are found as follows:—

Assuming the curve to be divided into 8 equal parts—

$$\begin{aligned}
\text{Offset 1 or } y_1 &= 4S(\tfrac{1}{8})^3 = \tfrac{1}{128}S \\
\text{,, 2 ,, } y_2 &= 4S(\tfrac{1}{4})^3 = \tfrac{1}{16}S \\
y_3 &= 4S(\tfrac{3}{8})^3 = \tfrac{27}{128}S \\
y_4 &= 4S(\tfrac{1}{2})^3 = \tfrac{1}{2}S \\
y_5 &= 4S(\tfrac{5}{8})^3 = \tfrac{125}{128}S \\
y_6 &= 4S(\tfrac{3}{4})^3 = \tfrac{27}{16}S \\
y_7 &= 4S(\tfrac{7}{8})^3 = \tfrac{343}{128}S \\
y_8 &= 4S(1)^3 = 4S
\end{aligned}$$

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In the field if the same lengths into which the spiral is subdivided be set off by marks along the tangent and the offsets measured from these marks, no appreciable inaccuracy will occur provided in putting in the station points the correct chainage is carefully maintained along the curve. This will only have the effect of pulling the end offsets a little away from the rectangular direction to the tangent, but not materially affecting the true positions of the points on the spiral. How little this is likely to affect results may be gathered from the fact that a spiral 300 feet long approaching a 7° curve (or one of 12·4 chains radius) has the correct point on the tangent from which the end offset (4S) should be measured at 299 feet instead of 300 feet from the B.S., the offset being 18·28 feet. [300 feet is long for a spiral to a 12·4 chain curve.]

Case B.—In this case offsets only require to be calculated for half the spiral or as far as the shift, the same offsets being used at equal

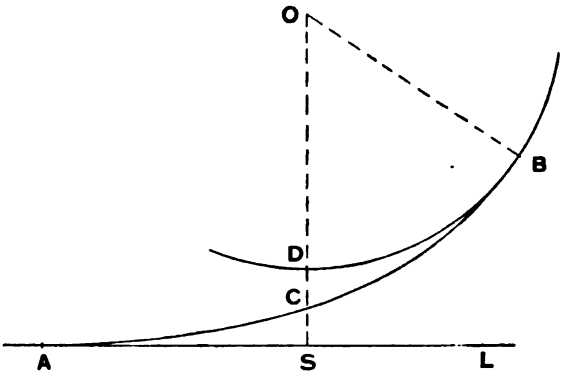

FIG. 44.

distances from the E.S. along the circular curve to determine the second half of the spiral back to the shift. This method is not so convenient for setting out the spiral on the ground, as it required the E.S. to be found on the circular curve and also the corresponding points at each offset on the circular curve. However, in preparing a spiral in the office this method is most useful and is performed as follows :—

The tangent ASL is first plotted, and at any point S a line drawn perpendicular to it (Fig. 44).

On this set off SD = shift and bisect SD in C, produce SD to O making DO = radius of circular curve. Then with O as centre and OD as radius draw circle DB. Now taking the half length of transition with a pair of compasses and placing the leg at C roughly locate the positions of A and B. Now sketch lightly in pencil the probable position of the spiral and measure its length from C by short divisions

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with a pair of dividers, half towards A and half towards B. In this way fix the actual positions of A and B, and divide the sketched curve AC into as many equal parts as half the number of offsets taken ; also CB into the same number of equal parts. The true position of the spiral is now found by measuring the offsets at each point of the sketched curve and correcting them by the calculated offsets. This operation is a very simple one and can be done quite quickly.

The curve is generally traced then and used as a trial spiral for improving some special portion of track.

Field Problems in Setting Out Transition Curves

In location work it is usual when the positions of the straights or tangents are pegged out on the ground to run in the circular curves.

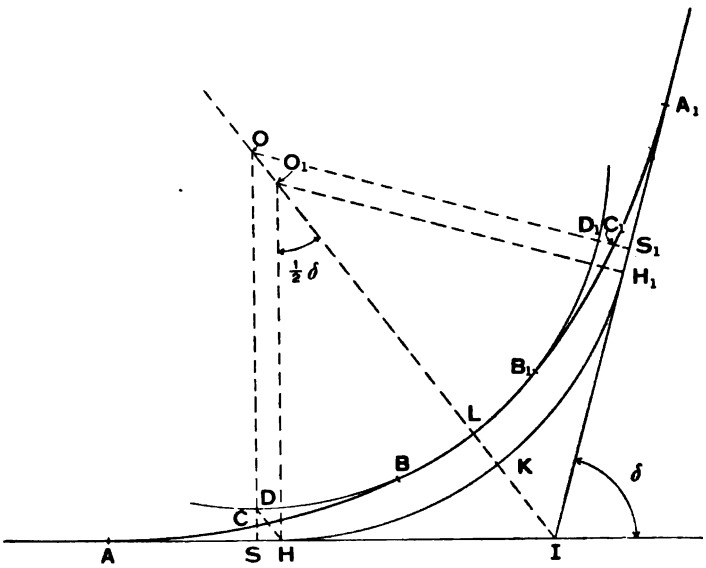

Fig 45.

Thus in Fig. 45, I is intersection of tangents, and HKH_1 the circular curve with centre O_1.

As it is rarely possible to alter the positions of the tangents, the circular curve HKH_1 must be moved into such a position as DBL in order to enable a spiral (ACB) to be connected with it. But as both ends of this circular curve will need spiralling it must be shifted symmetrically with respect to both tangents. Therefore the centre of the circle (O_1) must be moved back along the line (O_1I), bisecting the angle

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between the tangents. This has the effect of moving the circle into a position equally distant from both tangents.

Let O be the new position of O_1 and D the point corresponding to H. Then if the tangent to the circle at D was drawn, it would be parallel to AHI. When the length of spiral is decided upon, the shift SD is calculated. It then becomes necessary to find the points A and S by measurements back from I.

IH the subtangent to the circular curve is known $= r \tan \frac{1}{2}\delta$, also SD the shift $=$ S, therefore $SH = S \tan \frac{1}{2}\delta$ as HD is parallel to IO. Therefore $IS = r \tan \frac{1}{2}\delta + S \tan \frac{1}{2}\delta$ and $AI = \frac{l}{2} + (r + S) \tan \frac{1}{2}\delta$ as AS = AC approximately. Now that A is determined the spiral may be set out completely by deflection angles (or by tangent offsets). And thus having found B set the instrument over it and with twice the deflection angle (2θ) which was used at A to sight on B, set by the vernier, sight back on point A and clamp the lower limb. Now release the vernier plate and set it back to 0° 0′ 0″ when the line of sight will be along the common tangent to the spiral and the circular curve at point B. The rest of the circular curve can now be set out by deflection angles from this point, in the usual way, its length BLB_1 being equal to HKH_1 (already known for the circular curve) less the length of the transition spiral (AB). Thus point B_1 is found, and by setting the same deflection angle on the instrument at B_1 but on the opposite side of the zero, to observe B as was set at B to find B_1 and then releasing the vernier to read 0° 0′ 0″ the line of sight will be along the tangent at B_1. Now by deflecting the line of sight through 2θ the telescope should sight upon point A_1, which point could previously have been fixed by measuring $IA_1 = IA$, thus affording a good check upon the work. It is sometimes advisable to fix points D and D_1 when the instrument is set at point B or B_1. But it is unnecessary to chain along the circular curve to fix the points, as they may be determined by putting them upon the lines SC or S_1C_1 produced whilst observing with the correct deflection angles. These points also afford a useful check upon the accuracy of the work in setting out the spiral, as SC should equal CD. The second spiral $B_1C_1A_1$ can be set out by deflection angles from point B_1 or from point A_1 as previously shown.

Transition Curves Connecting Two Circular Curves

Take the case of two curves turning in the same direction. In order to solve this problem it is necessary to find a relation connecting the radii of curvature at two points on the transition spiral with the length of curve between them. From equation of spiral (3) (p. 126) $\lambda = m\sqrt{\phi}$, and therefore

$$\rho = \frac{d\lambda}{d\phi} = \frac{m}{2\sqrt{\phi}}$$

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$\therefore$ (in Fig. 46) if r_1 and r_2 are radii of curvature at B_1 and B_2 respectively, then

$$r_1 = \frac{m}{2\sqrt{a_1}}, \text{ and } r_2 = \frac{m}{2\sqrt{a_2}}$$

$$\therefore \sqrt{a_1} - \sqrt{a_2} = \frac{m}{2}\left(\frac{1}{r_1} - \frac{1}{r_2}\right) = \frac{m(r_2 - r_1)}{2r_1r_2} \quad . \quad . \quad . \quad . \quad (t)$$

Similarly it may be shown that $\sqrt{a_1} + \sqrt{a_2} = \dfrac{m(r_2 + r_1)}{2r_1r_2}$

$\therefore$ taking the product of this equation with (t),

$$a_1 - a_2 = \frac{m^2(r_2^2 - r_1^2)}{4r_1^2r_2^2}$$

$$\therefore a = a - a_2 = \frac{m^2(r_2^2 - r_1^2)}{4r_1^2r_2^2}$$

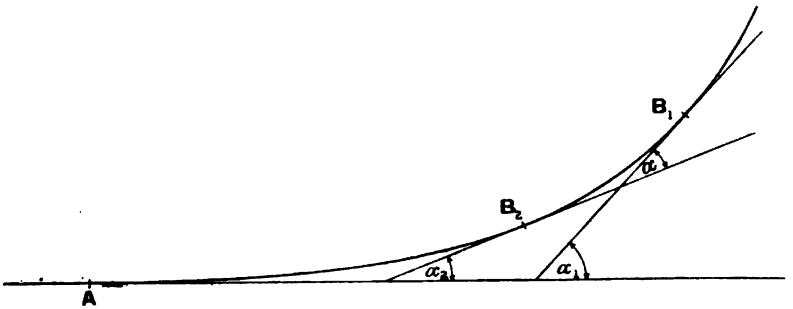

FIG. 46.

But if l_1 = length of spiral AB_1 and $l_2 = AB_2$, then l the portion of curve between B_1 and $B_2 = l_1 - l_2$, also from equation (c) $l_1 = m\sqrt{a_1}$ and $l_2 = m\sqrt{a_2}$

$$\therefore l = l_1 - l_2 = m(\sqrt{a_1} - \sqrt{a_2}) = \frac{m^2(r_2 - r_1)}{2r_1r_2}$$

$$\therefore \frac{a}{l} = \frac{r_2 + r_1}{2r_1r_2} \quad \text{or} \quad \frac{l}{a} = \frac{2r_1r_2}{r_2 + r_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (u)$$

which is a relation between the radii of curvature at points B_1 and B_2 in terms of the length of spiral between the points and the angle through which the curve has turned in that length.

In Fig. 46, a_1 and a_2 are much larger than those which occur in practice.

Now, if DB_1, in Fig. 47, be circular curve with centre O_1 and B_2S circular curve with centre O_2, which it is desired to connect by spiral B_2CB_1, and if the centres O_2 and O_1 be joined and produced to intersect the circles in D and S, then DS = S, the shift of the circles being the shortest distance between the curves.

In the case of compound curves, where two curves of different radii join at their common tangent point, their centres of curvature are on the line at right angles to this tangent, and the displacement or shift which it is necessary to make in order to enable the transition to be inserted, should be made along this line if the curves are not to be displaced in direction. It is interesting to note from the theoretical investigation for the transition for two such curves that the shift actually occurs along this line.

Now, a_1 is central angle for arc DB_1 and a_2 is central angle for arc B_2S, and a the angle between tangents at B_1 and B_2. Draw line O_1K parallel to O_2B_2.

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Then angles at K and B_1 are right angles, and $\alpha = \alpha_1 + \alpha_2$. Also $B_2CB_1 = l$, and as S is small compared with l, l may be taken equal to B_2S plus DB_1

$$\therefore l = r_2\alpha_2 + r_1\alpha_1$$

But $$\alpha = \alpha_2 + \alpha_1$$

$$\therefore l = r_1\alpha_1 + r_2(\alpha - \alpha_1)$$

But from equation (*u*) $l = \dfrac{2r_1r_2\alpha}{r_2 + r_1}$

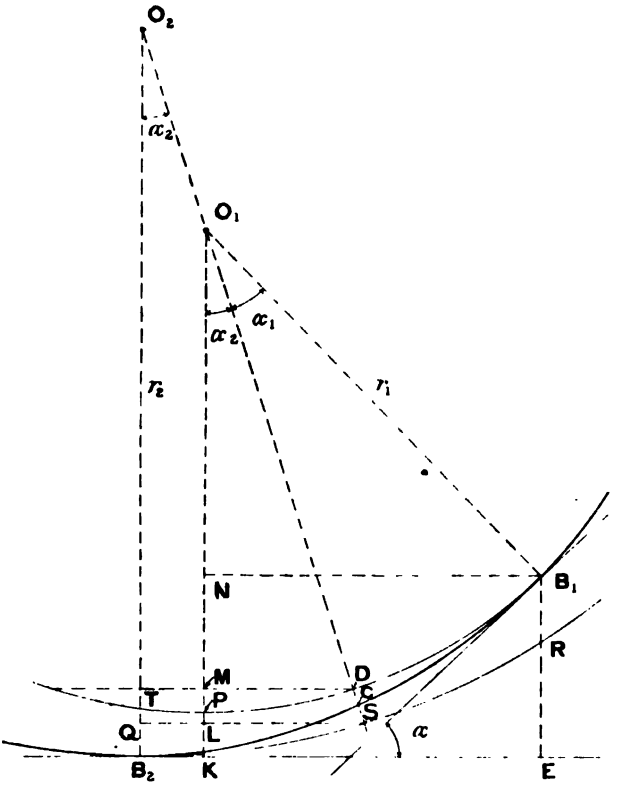

FIG. 47.

Equating these two value of l gives

$$\alpha_1 = \frac{r_2\alpha}{r_2 + r_1}$$

Similarly it may be shown by eliminating α_1 from the first equation that

$$\alpha_2 = \frac{r_1\alpha}{r_2 + r_1}$$

$$\therefore \frac{\alpha_1}{r_2} = \frac{\alpha_2}{r_1} \quad \text{or } \alpha_1r_1 = \alpha_2r_2 = \frac{r_1r_2\alpha}{r_2 + r_1} = \frac{l}{2}$$

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Therefore line SD bisects B_2CB_1 in C, or half of the transition curve must be on each side of the "shift" or shortest line joining the circles.

To find SD or the amount of the shift, draw B_1E perpendicular to the tangent at B_2, and draw SL, DM, and B_1N parallel to the tangent B_2E. Then $B_1E = KL + LM + PN - PM$; and expressing KL, PN, and PM as in the method of versines, we have

$$KL = QB_2 = \frac{\left(\frac{l}{2}\right)^2}{2r_2}$$

and $PN = \frac{r_1^2(a_1 + a_2)^2}{2r_1}$ and $PM = \frac{r_1^2 a_2^2}{2r_1}$ and $LM = S$ the shift.

These are true, on the assumption that a_1 and a_2 are very small angles, which always occurs in practice.

$$\therefore B_1E = \frac{\left(\frac{l}{2}\right)^2}{2r_2} + S + \frac{r_1^2(a_1 + a_2)^2}{2r_1} - \frac{r_1^2 a_2^2}{2r_1}$$

$$= \frac{l^2}{8r_2} + \frac{r_1^2 a^2}{2r_1} - \frac{r_1^2 a_2^2}{2r_1} + S$$

$$= \frac{l^2}{8r_2} + \frac{r_1^2}{2r_1} \cdot \frac{(r_1 + r_2)^2 l^2}{4r_1^2 r_2^2} - \frac{r_1^2}{2r_1} \cdot \frac{l^2}{4r_2^2} + S$$

$$= \frac{l^2}{8r_1^2 r_2^2}(r_1 r_2^2 + 3r_1^2 r_2) + S = \frac{l^2}{8r_1 r_2}(r_2 + 3r_1) + S \quad . \quad (v)$$

It is now necessary to get another expression for B_1E. This can be obtained from the equation of the curve by finding the value of y when B_2 is the origin of co-ordinates and B_2E and B_2O_2 axes of x and y. First taking the general equation with origin A where $\rho = \infty$, as in Fig. 40, it may be put in the form— $l_2 + l = n\sqrt{(a_2 + a)}$ where l_2 and a_2 refer to a portion of the curve of constant length, at whose extremities a has the values zero and a_2, we have

$$\frac{dl}{da} = \rho = \frac{n^2}{2(l_2 + l)}$$

Now, $\rho = r_2$ when $l = o$, also $\rho = r_1$ when $a = a_1$, and $l = B_2B_1$ or l^1

$$\therefore r_2 = \frac{n^2}{2l_2} \text{ and } r_1 = \frac{n^2}{2(l_2 + l^1)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (w)$$

Now, referring the curve to the rectangular co-ordinates. At the origin B_2, $\rho = r_2$.

Taking an element of the curve ($l_2 + l = n\sqrt{(a_2 + a)}$)—

$$dx = \frac{n \cos a}{2\sqrt{(a_2 + a)}} \cdot da$$

Similarly

$$dy = \frac{n \sin a}{2\sqrt{(a_2 + a)}} \cdot da$$

Now, when a is small

$$dy = \frac{n \,.\, a \,.\, da}{2\sqrt{(a_2 + a)}}$$

$$\therefore y = \frac{n}{2}\int \frac{a da}{\sqrt{(a_2 + a)}} = \frac{n}{2}\left[\tfrac{2}{3}(a_2 + a)^{\frac{3}{2}} - 2a_2\sqrt{(a_2 + a)}\right] + \text{constant}$$

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For the constant, when $a = o$, $y = o$

$$\therefore \frac{n}{2}\left[\tfrac{2}{3}a_2^{\frac{3}{2}} - 2a_2^{3}\right] + \text{constant} = o$$

$$\text{or constant} = \frac{n}{2}\cdot\tfrac{4}{3}a_2^{\frac{3}{2}} = \tfrac{2}{3}na_2^{\frac{3}{2}}$$

$$\therefore y = \frac{n}{2}\left[\tfrac{2}{3}(a_2 + a)^{\frac{3}{2}} - 2a_2\sqrt{(a_2 + a)}\right] + \tfrac{2}{3}na_2^{\frac{3}{2}} \quad . \quad . \quad . \quad (x)$$

Similarly it can be shown that $x = n\sqrt{(a_2 + a)} - n\sqrt{a_2}$

$$\therefore \sqrt{(a_2 + a)} = \frac{x + n\sqrt{a_2}}{n}$$

can be substituted in (x), which becomes

$$y = \frac{n}{2}\left[\frac{2}{3}\left(\frac{x + n\sqrt{a_2}}{n}\right) - 2a_2\left(\frac{x + n\sqrt{a_2}}{n}\right)\right] + \tfrac{2}{3}na_2^{\frac{3}{2}}$$

This reduces to

$$y = \frac{x^3}{3n^2} + \frac{x^2\sqrt{a_2}}{n}$$

But in general equation $l = o$ when $a = o$

$$\therefore l_2 = n\sqrt{a_2}$$

$$\therefore y = \frac{x^3}{3n^2} + \frac{l_2x^2}{n^2}$$

But from (w)

$$\frac{2l_2}{n^2} + \frac{2l^1}{n^2} = \frac{1}{r_1} \quad \text{and} \quad \frac{2l_2}{n^2} = \frac{1}{r_2}$$

$$\therefore n^2 = \frac{2r_1r_2l^1}{r_2 - r_1}$$

$$\therefore y = \frac{x^3(r_2 - r_1)}{6r_1r_2l^1} + \frac{x^2}{2r_2}$$

From this equation another expression for B_1E may be obtained by putting $x = l^1$ and then changing l^1 into l. (l^1 has only been used instead of l to distinguish it from a variable.)

$$\therefore B_1E = \frac{l^2(r_2 - r_1)}{6r_1r_2} + \frac{l^2}{2r_2} = \frac{l^2}{6r_1r_2}(r_2 + 2r_1)$$

Therefore from (v)

$$S = \frac{l^2}{6r_1r_2}(r_2 + 2r_1) - \frac{l^2}{8r_1r_2}(r_2 + 3r_1)$$

$$\therefore S = \frac{l^2}{24r_1r_2}(r_2 - r_1) = \frac{l^2}{24}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)^*$$

Obviously, when $r_2 = \infty$ we get the expression for the shift $\frac{l^2}{24r_1}$ in the case of a tangent and one circular curve as on p. 129.

Again, if in above equation for y x be put equal to $\frac{l^1}{2}$ and l^1 changed to l, we get $SC = \frac{l^2}{48}\left(\frac{1}{r_1} - \frac{1}{r_2}\right) = \frac{1}{2}S$.

Therefore both the transition curve and the shift are bisected at C.

* This value is also given by Mr. James Glover in his paper entitled "Transition Curves for Railways," *Minutes of Proceedings*, vol. cxl., Inst. C.E.

This formula for the shift is the same as given in Rankine's "Civil Engineering," derived from Mr. Froude's curve of adjustment.

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Again, the amount of divergence B_1R of the spiral from the circular curve at B_1 = BE − ER (approximately) and ER = $\frac{l^2}{2r_2}$

$$\therefore B_1R = \frac{l^2}{6r_1r_2}(r_2 + 2r_1) - \frac{l^2}{2r_2} = \frac{l^2}{6r_1r_2}(r_2 - r_1) = \frac{l^2}{6}\left(\frac{1}{r_1} - \frac{1}{r_2}\right) = 4S$$

But $B_2T = B_1R$ (approx.) as both T and B_1 are symmetrically placed with regard to the line O_2O_1S. Therefore the transition curve deviates from the circle of larger radius at the same rate starting from one end of it as it does from the circle of smaller radius starting from the other end. It is also obvious from this that the shift must be bisected by the transition curve.

It may be observed from above value of S that S and l are dependent upon each other; that is, provided the circular curves are not given in position but only their radii fixed.

If the circular curves are given in position the "shift" is fixed, and therefore l must be determined from the equation

$$S = \frac{l^2}{24}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)$$

If l as found by this method proves unsuitable on account of the maximum speed to be allowed for on the road, it may be necessary to change the radius of one of the curves, if it is impossible to alter the shift.

If the length of transition is calculated by the formula $l = \sqrt{R}$, this R must refer to the circular curve of smaller radius as the matter of the maximum allowable speed on the curve is involved in it, and if the lengths of transition for the two radii were calculated by this formula, and the lesser deducted from the greater, the length so obtained would not be sufficiently long to enable the necessary superelevation to be gained.

This is natural, as a transition which was efficient in the case of one circular curve could not be so in the case of the other curve. But as the speed cannot exceed that permissible on the sharper curve, that transition should be selected which is suitable for the sharper curvature.

It will be observed in putting in transitions in these cases of direct curves how small the shift is. Taking a case with as wide a deviation in curvature as 50 chains and 25 chains the shift is only $4\frac{1}{8}$ inches, but generally the variation is not even so great and consequently the shift is less. There are few cases of compound curves in practice deflecting in the same direction where it is not possible to insert a transition curve for their improvement on account of the shift being so small. It may also be noticed from the formula for the shift that it varies as the square of the length of the transition. Accordingly if the transition is shortened even a small amount it takes off a much larger proportion from the length of the shift. It is not always necessary or indeed possible to insert a transition curve between two circular curves. If the smaller radius is only 15 per cent. less than the larger the shift is $\frac{1}{8}$th of an inch, which is almost too small for setting the curve out on the ground. Similarly :—

A 20 per cent. reduction will give a shift				$\frac{1}{8}$-inch long.
„ 25	„	„	„	$\frac{1}{2}$ „
„ 31	„	„	„	1 „
„ 39	„	„	„	2 „
„ 50	„	„	„	$4\frac{1}{8}$ „

The usual method in improving these curves is to shift the sharper curve in along the common line of centres an amount equal to the shift. This will have the effect of removing the other end of the curve from its tangent point in the straight, but as that end will also need a transition curve the shift will be provided for it by the one movement of the circular curve.

The "setting out" of this transition is simple. Having calculated the length

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of transition, determine by the formula the amount of the shift and set this off at right angles to the common tangent at the tangent point, and setting a theodolite over this point set out the new circular curve by chords of a length the same as required for the transition curve. The transition curve is then set out from these points by the ordinary offsets of the transition spiral used, the first half of the transition being set out in a similar manner by offsets from points in the first arc of large radius. If more convenient the new circular curve can be set out by offsets from the common tangent or, indeed, the transition may be set out direct from the tangent by using offsets which are the difference between the offsets mentioned above.

If instead of a compound curve of the kind here referred to a short length of straight exists between the two curves, it is not possible to replace this by one transition curve unless the smaller circle can be shifted to a position where it would be inside the arc of the larger curve produced. This generally involves much too large a shift for practical purposes. However, it should be possible to spiral both curves, or at any rate the sharper one, in the ordinary way. In doing so the length of transition should be determined for the sharper curve by $L = \sqrt{R}$ and a portion of this transition be used for the flatter curve. If done in the reverse order much longer transition curves would be used than were needed for good running. It may be remarked that the custom of super-elevating curves according to their curvature is not always a scientific operation. In this case the sharper curve has the superelevation suitable to the maximum speed to be attained in running on it, but the flatter curve will only have the superelevation needed for this speed, and consequently a much shorter transition curve than if calculated for the maximum speed which the flatter curvature would justify. Consequently if a number of curves of varying curvature occur at close intervals along a portion of line where there is little variation in the gradient the length of transition for each of these curves if determined by the above formula—and it is a formula to be recommended in most cases—should be found for that of smallest radius first and portions of this transition used for the other curves according to their curvatures.

By this means the best running will be obtained over the whole of this portion of line, as the rate of change of acceleration in approaching and leaving each curve will be the same.

Reverse Curves

In spiralling reverse curves the spirals approaching each curve should belong to the same spiral curve, *i.e.* should change their curvature at the same rate. This is necessary in order that at the point of reverse no sudden change in motion may be felt in the passage of a train from one circular curve to the other.

Fig. 48 shows a spiral curve AL_1B_1 and a shorter portion of the same used for the reverse curve AL_2B_2.

The radius of curvature at B_1, r_1 is less than r_2, the radius of curvature at B_2. That is, the circular curve with centre O_1 being sharper than that with centre O_2, requires the spiral to be extended to B_1 till its curvature is equal to that of the circle of radius r_1, whereas a shorter portion of the same spiral serves the purpose of approaching the circular curve (r_2). The straight line E_2ACE_1 is tangent to the spirals at their common origin A. Also O_1O_2 intersects this line in C and the circular curves in D and S. The original positions of the circular curves is shown by CP and CQ with their common tangent CR at C. If perpendiculars be dropped on E_2AE_1 from O_1 and O_2, namely O_1M and O_2N, these intersect the circles in their original positions in P and Q and in the positions into which they have been shifted for spiralling purposes in F and G. Now, MF is the shift for spiral AB, and NG that for spiral AB_2. Therefore if $l_1 = AB_1$ and $l_2 = AB_2$, $\frac{1}{2}l_1 = AL_1 = FB_1$. Also $\frac{1}{2}l_2 = AL_2 = GB_2$. The angle subtended by $\frac{1}{2}l$ at the centre O_1 is α_1 and that by $\frac{1}{2}l_2$ at O_2 is α_2. Also

$$\frac{L}{}$$

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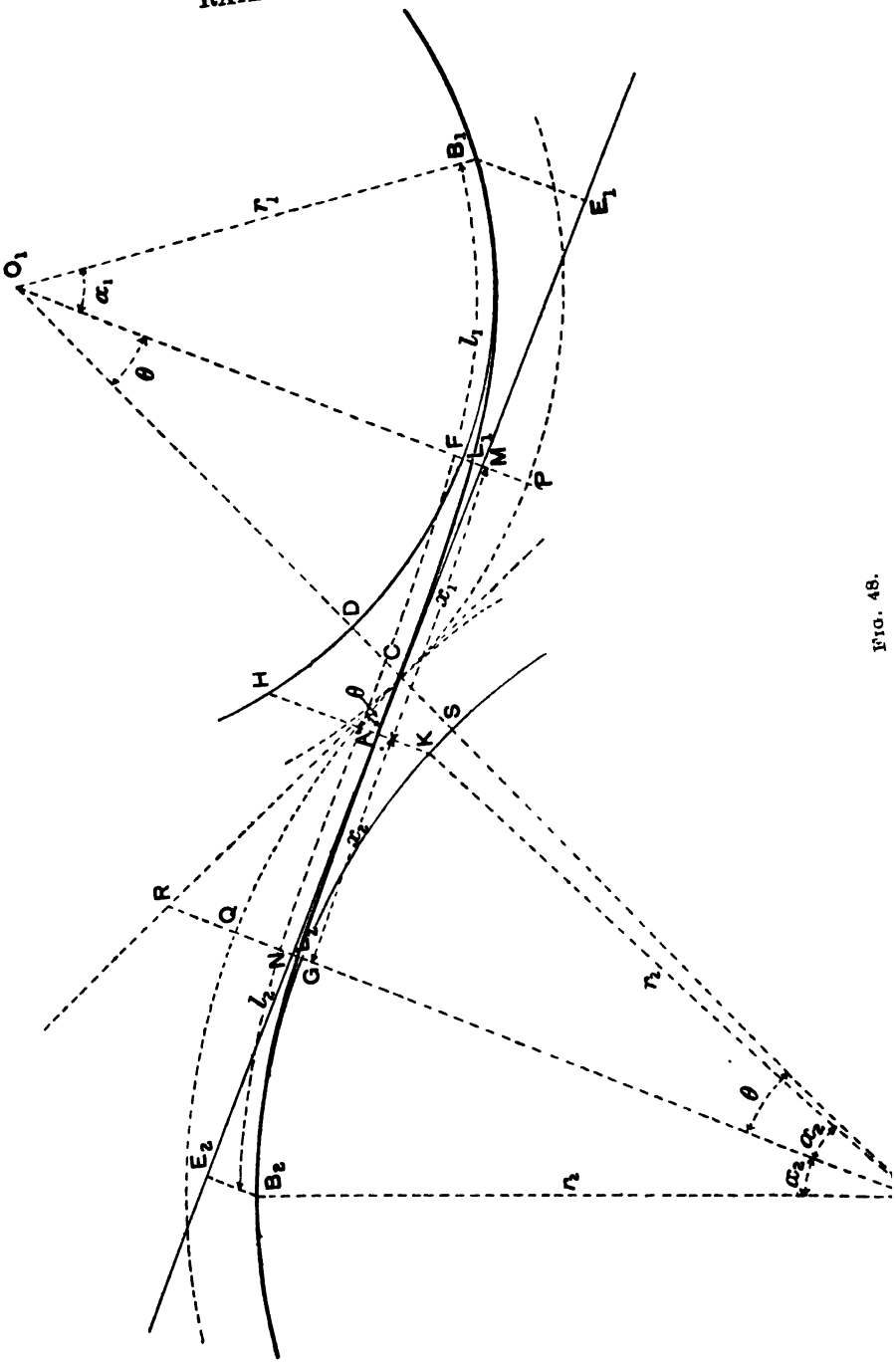

FIG. 48.

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the angles made by O_1O_2 with the perpendiculars on the common tangent are equal and called θ. It may be observed that the angle RCE_2 made by the two common tangents is also equal to θ.

Now, from the equation of the curve $l_1 = m\sqrt{a_1} = \frac{m^2}{2} \cdot \frac{1}{r_1}$

Similarly $l_2 = \frac{m^2}{2} \cdot \frac{1}{r_2}$ $\therefore \frac{l_1}{l_2} = \frac{r_2}{r_1}$

$\therefore AM : AN :: r_2 : r_1$

But in the similar triangles CO_1M and CO_2N

$CM : CN :: r_1 : r_2$ approximately. $\therefore CM : CN :: AN : AM.$

But $CM + CN = AN + AM.$ $\therefore CM = AN = \frac{1}{2}l_2$ and $CN = AM = \frac{1}{2}l_1$. $\therefore AC = \frac{1}{2}(l_1 - l_2)$, which means that the origin of the spirals A is shifted along the tangent from the common tangent point (C) of the original circular curves in the direction of that of larger radius by an amount equal to half the difference of the lengths of the spirals.

It should be observed that C is the centre point of the total spiral line B_1CB_2. Thus $CB_1 = CB_2 = \frac{1}{2}(l_1 + l_2)$.

To find the amounts the original curves need shifting, namely SC and DC :—

$$SC = QG = QN + NG \quad \text{and} \quad DC = PF = PM + MF$$

Now, $QN = \frac{(NC)^2}{2r_2} = \frac{l_1^2}{8r_2}$ and $NG = \text{shift} = S_2 = \frac{l_2^2}{24r_2}$

$$\therefore SC = \frac{l_1^2}{8r_2} + \frac{l_2^2}{24r_2} = \frac{3l_1^2 + l_2^2}{24r_2}$$

Similarly $DC = \frac{l_2^2}{8r_1} + \frac{l_1^2}{24r_1} = \frac{3l_2^2 + l_1^2}{24r_1}$

These are the amounts by which the respective circles must be shifted back along the line O_1O_2.

Now, since $\frac{l_1}{l_2} = \frac{r_2}{r_1}$ $\frac{\frac{1}{l_1} + \frac{1}{l_2}}{\frac{1}{l_2}} = \frac{\frac{1}{r_1} + \frac{1}{r_2}}{\frac{1}{r_1}}$

$\therefore$ the value for the total shift

$$SD = SC + CD = \frac{l_1^2}{8r_2} + \frac{l_2^2}{24r_2} + \frac{l_2^2}{8r_1} + \frac{l_1^2}{24r_1}$$

and may be expressed as

$$\frac{l_1^2(r_2^3 + 3r_1r_2^2 + 3r_1^2r_2 + r_1^3)}{24r_2^3r_1} = \frac{l_1^2(r_1 + r_2)^3}{24r_1r_2^3}$$

by substituting for l_2 its value $l_1 \cdot \frac{r_1}{r_2}$.

$$\therefore SD = \frac{l_1^2(r_1 + r_2)^3}{24r_1r_2^3} = \frac{l_1^2r_1^2}{24}\left(\frac{1}{r_1} + \frac{1}{r_2}\right)^3 = \frac{l_1^2}{24}\frac{\left(\frac{1}{l_1} + \frac{1}{l_2}\right)^2}{\frac{1}{l_2^2}} \cdot \left(\frac{1}{r_1} + \frac{1}{r_2}\right)$$

$$= \frac{l_1^2l_2^2}{24} \cdot \left(\frac{l_1 + l_2}{l_1l_2}\right)^2 \left(\frac{1}{r_1} + \frac{1}{r_2}\right) = \frac{(l_1 + l_2)^2}{24} \cdot \left(\frac{1}{r_1} + \frac{1}{r_2}\right)$$

This is a similar expression to that obtained on p. 143 for curves curving

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in the same direction, the sign there being − instead of + and the l there being equivalent to $l_1 - l_2$ here.

In determining the direction of the tangent line E_1CE_2 from the original common tangent RC the angle θ is given by

$$\text{Sin } \theta = \frac{l_1}{2r_2} = \frac{l_2}{2r_1}$$

A closer approximation to the value of θ is generally obtained by taking

$$\tan \theta = \frac{l_1}{2r_2} = \frac{l_2}{2r_1}$$

It will be seen from the above solution that the common custom of railways to use the point of common tangency of the circular curves as the point of reverse for the transition curves cannot be adopted if it is the intention to make the rate of change of acceleration continuous on the transition.

Unlike the transition between two curves turning in the same direction, the shift of the circular curves in the case of reverse curves is a rather large quantity, and in many cases in practice would involve shifting the existing roads too much for the space available. Consequently it is recommended to put in a sharper portion of circular curve for each curve at its end adjacent to the common tangent in the manner described on p. 118 and thus leave the main part of each curve unaltered, but pull in the ends of the curves by slightly reducing the radii so as to effect the required shift.

It has been suggested by M. Lafut that in the cases where the shift of the circular curve is greater than can be permitted on an existing line, the quartic parabola should be used. This curve, however, not being a true transition curve, will produce inferior running to that obtained on the transition spiral.

Improving an Existing Curve

The problem frequently occurs that an existing circular curve, GJT (in Fig. 49), running directly off a tangent LG needs easement without the possibility of altering the main part of the circular curve. This may be effected by sharpening a portion of the circular curve as it approaches the tangent, as JBD in figure, whose centre is O_1 instead of O, and then inserting a transition spiral ACB to connect the tangent with this circular curve.

It has been shown by Mr. W. H. Shortt [1] that it is unnecessary to connect two circular curves curving in the same direction by a transition curve if the radius of one does not differ from the other by more than 15 per cent. But that any pair of curves conforming with this requirement must have the length of each not less than one-sixth of the square root of its radius in chains.

Thus, in above figure, if circle GJT be of 35 chains radius, and it is decided to sharpen the end JG to one of 30 chains radius (JBI), the change here being less than 15 per cent. is permissible. Then

[1] *Minutes of Proceedings*, Inst. C.E., vol. clxxvi., pp. 117 and 118.

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having decided upon the length of transition the shift SD can be calculated. Or taking

$$l = \sqrt{r}, \quad S = 2 \text{ ft. } 9 \text{ inches.}$$

Now, taking O as centre and $OO_1 = r - r_1$, or, say, 5 chains as radius, describe circle HO_1. On GO the perpendicular on tangent, measure off HF = "Shift" and draw FO_1 parallel to tangent till it cuts circle in O_1. Join OO_1 and produce to J. Draw perpendicular O_1DS on tangent produced—

$$\therefore O_1S = FG = HG + S = O_1J + S = O_1D + S$$
$$\therefore DS = S \text{ (the shift).}$$

$\therefore$ Circle with centre O_1 and radius O_1J or r_1 will have the required shift and also be of required radius. Then bisecting DS in C and setting off a length equal to $\frac{l}{2}$ in both directions, CB and CA, the ends of the spiral are determined.

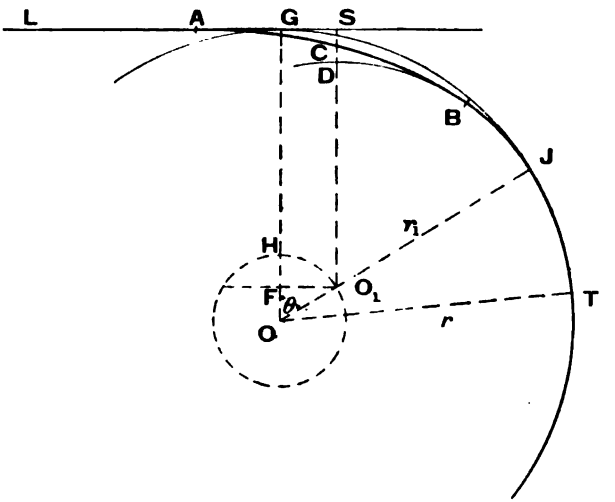

FIG. 49.

If BJ is not less than one-sixth of $\sqrt{r_1}$, or in this case, where $r_1 = 30$ chains, ·9 chains, the conditions will be completely satisfied.

To Set Out Curve on the Ground.—G being the B.C. is fixed on the ground and J can be found by calculating θ and measuring the distance subtended by θ along the curve—

$$\cos\theta = \frac{OF}{OO_1} = \frac{r - r_1 - S}{r - r_1} = \text{known}$$

Also $$GS = FO_1 = OO_1 \sin\theta = (r - r_1)\sin\theta$$

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Therefore J the common point of the two circular curves being found, the curve JBD can be run in by deflection angles from this point. Also S is found, knowing GS and D can be set out by an offset = S and should be on circular curve JBD.

Next, point A is found by measuring back from S a distance $\frac{1}{2}l$. Then the spiral can be set out from A by deflection angles and the accuracy of the curve determined by observing if it passes through C and B, points already fixed.

Practical Examples of Transition Curves. Connecting two Circular Curves

Curves Curving in the Same Direction.—Two circular curves of 30 (r_1) and 40 (r_2) chains radius respectively curving in the same direction and forming a compound curve have their common tangent at B (Fig. 50). It is necessary to shift each curve in towards its centre to enable the spiralling to be performed. The main tangents or straights QAFL and MEN are assumed to be immovable (this is generally the case in practice). The circular curves must be moved in so as to enable not alone the spiral B_1CB_2 connecting the two curves to be affected, but also that their other extremities, in their new positions, may be capable of being connected with their respective straights by spirals.

Assuming the superelevation for the 30 chains radius curve to be ·45 feet and that for the 40 chains ·3 feet and that the rate at which the cant is applied is 1 inch in 50 feet or 1 in 600 (a rule commonly used in practice), then the length of transition curve (l_1) for the former is ·45 × 600 = 270 feet and that for the latter (l_2) is ·3 × 600 = 180 feet. These should be the lengths of spirals used at the extreme ends of the curves, whilst near the centre B that portion of a 270-foot spiral between its 180-foot point and its 270-foot point (or a 90-foot length) is used for connecting the two curves. This can also be seen from the difference in cants, for ·45 − ·3 = ·15 and ·15 × 600 = 90 feet. Next consider the shifts or distances which each curve must be moved in on the line of their common centres BO_1O_2. The distance SD (S) between the circular curves is given by formula

$$S = \frac{(l_1 - l_2)^2}{24}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)$$

on p. 143 where $l = l_1 - l_2$.

Also the shifts S_1 and S_2 for the other ends of the circular curves are given by

$$S_1 = \frac{l_1^2}{24r_1} \text{ and } S_2 = \frac{l_2^2}{24r_2}$$

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The values of these in the present example are :—

$$S_1 = \frac{270 \times 270}{24 \times 30 \times 66} = 1{\cdot}53 \text{ feet} = 18{\cdot}4 \text{ inches}$$

$$S_2 = \frac{180 \times 180}{24 \times 40 \times 66} = {\cdot}51 \text{ feet} = 6{\cdot}13 \text{ inches}$$

$$S = \frac{(270 - 180)^2}{24}\left(\frac{1}{30} - \frac{1}{40}\right) = {\cdot}51 \text{ inches}$$

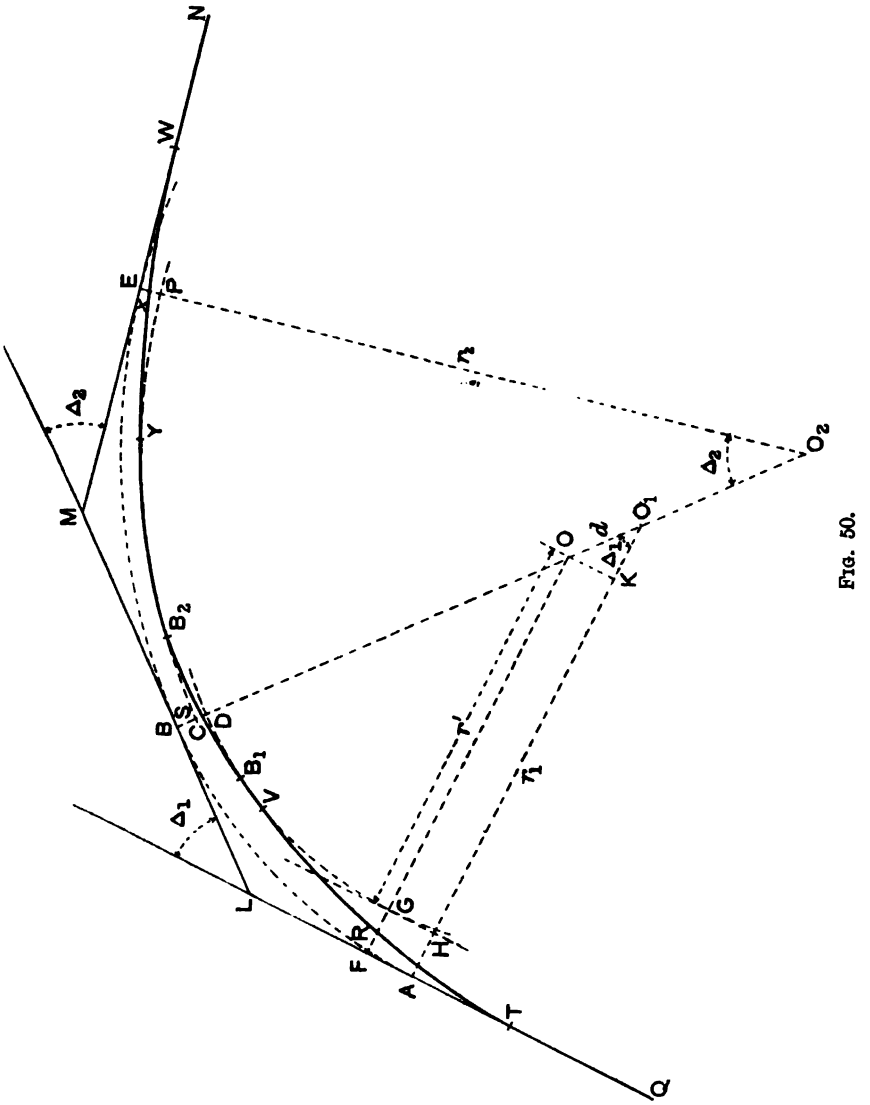

FIG. 50.

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As the 40-chain curve needs shifting only 6·13 inches this can be done by shifting the station pegs (or centre pegs) in 6·1 inches, that one at point B being moved to S. Then a further length SD = ·51 inches needs setting out on this line. This fixes the point D on the new position of the 30-chain curve. The other extremity of this curve needs to be removed to a distance S_1 from tangent QAFL as at G where FG = AH = S_1. Probably the simplest way to effect this is to reduce the radius r_1 a little making it r' in figure, its centre being at O. This reduction is generally so small that the alteration to the curvature is negligible. In the figure GH and OK are at right angles to AO_1 and OGF is parallel to AO_1

But it can be seen that—

$r_1 = AH + HK + KO_1 = S_1 + r' + d \cos \triangle_1$, where $d = OO'$ and $\triangle_1$ is the central angle for the sharper circular curve and therefore equal to the external angle of the subtangents meeting at L.

Again $r_1 = BS + SD + DO + OO_1 = S_2 + S + r' + d$

$\therefore S_2 + S + r' + d = S_1 + r' + d \cos \triangle_1$

$\therefore d\,(1 - \cos \triangle_1) = S_1 - (S_2 + S)$

If in this case $\triangle_1$ has the value 20°—

Then $d = \dfrac{S_1 - (S_2 + S)}{1 - \cos 20^\circ} = \dfrac{18{\cdot}4 - (6{\cdot}13 - {\cdot}51)}{{\cdot}06031} = 195'' = 16{\cdot}25$ feet

But $r' = r_1 - S_1 - d \cos \triangle_1 = 30 \times 66 - 1{\cdot}53 - 15{\cdot}27 = 1963{\cdot}2$

$= 29{\cdot}75$ chains

This reduction in the radius though much facilitating the operation of setting out the curves for spiralling is not of serious moment as regards the speed being used on this curve as the approaching spiral from Q end will diminish oscillations, also its alteration of the position of B_1 is almost infinitesimal and certainly could not be set out on the ground. Now A being the original tangent point, F must be found so that FG = S_1 can be set off at it, in order to fix the position of G which is on the new circular curve. Also R the middle point of FG can be set out, for this is the middle point of the spiral l_1 (TRV in figure).

Now AF = OK = $d \sin \triangle_1 = 16{\cdot}25' \times \sin 20^\circ = 5{\cdot}56$ feet. Therefore measuring from the tangent point A 5·56 feet along the subtangent AL gives the point F at which a rectangular offset of length 1·53 feet gives the position of G. The circular curve GVB_1D can now be set out by deflection angles from G or D, the deflection angles being calculated for a 29·75 chain curve. This angle for each chain length = $\dfrac{1718{\cdot}87}{29{\cdot}75}$ minutes or 57·77 minutes.

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Therefore assuming GV $= \frac{1}{2}l_1 = \frac{1}{2}(270) = 135$ feet, the point V can be set out at the same time by turning the angle for $\frac{135}{66}$ chains $= 2\frac{1}{22}$ chains. Similarly the point B_1 can be set out by deducting from the total length of curve GD for a central angle $\Delta_1(20°)$ the length DB_1 or 45 feet.

Again, measuring from F a length FT $= \frac{1}{2}l_1 = 135$ feet, gives the position of the B.S. of spiral TRV.

Similarly from E the tangent point of curve r_2 measure off EW, and from P (EP $= S_2 = \cdot 51$ feet) PY, each equal to l_2 or 90 feet. Then we have on the ground the beginnings and ends of the three transition curves.

If the deflection angles for the longest of these, namely, l_1 are prepared they can be used for all three curves.

Reverse Curves.—Two circular curves of 30 chains (r_1) and 40 chains (r_2) radius curve in opposite directions from a point of common tangency.

Their superelevations are ·45 feet and ·3 feet respectively, therefore the lengths of their respective transition curves are 270 feet and 180 feet, as in the previous example. But the transition connecting the reverse curves has its length determined by ·45 + ·3 = ·75 feet, and equals ·75 × 600 = 450 feet. The lengths l_1 and l_2 of the respective spirals are 270 feet and 180 feet. The common tangent NACM, at the point A from which both spirals branch (see Fig. 51), is found by deflecting an angle ϕ at C from the common tangent to the reverse circular curves QCP (see p. 148).

$$\sin \phi = \frac{l_1}{2r_2} = \frac{270}{2 \times 40 \times 66} = \cdot 05113. \quad \therefore \phi = 2° 56'$$

Thus having deflected through 2° 56′ set out on this line the points N, A and M at distances 270 feet (270 − 180) feet or 90 feet, and 180 feet from C.

Also set out points D and S on the line at right angles to the original common tangent. These shifts s' and s'' are given by

$$s' = \frac{3l_2^2 + l_1^2}{24r_1} = \frac{3 \times (180)^2 + (270)^2}{24 \times 30 \times 66} = 3{\cdot}58 \text{ ft.}$$

$$s'' = \frac{3l_1^2 + l_2^2}{24r_2} = \frac{3(270)^2 + (180)^2}{24 \times 40 \times 66} = 3{\cdot}96 \text{ ft.}$$

In shifting the circular curves inwards to these respective points D and S, their other extremities cease to touch the straights or tangents. This will not matter in the case of either which touches a line, as VU in Fig. 51, parallel to the original tangent, and at a distance from it, on the inner side, equal to the shift of the spiral connecting the circular curve with the straight. But as this rarely

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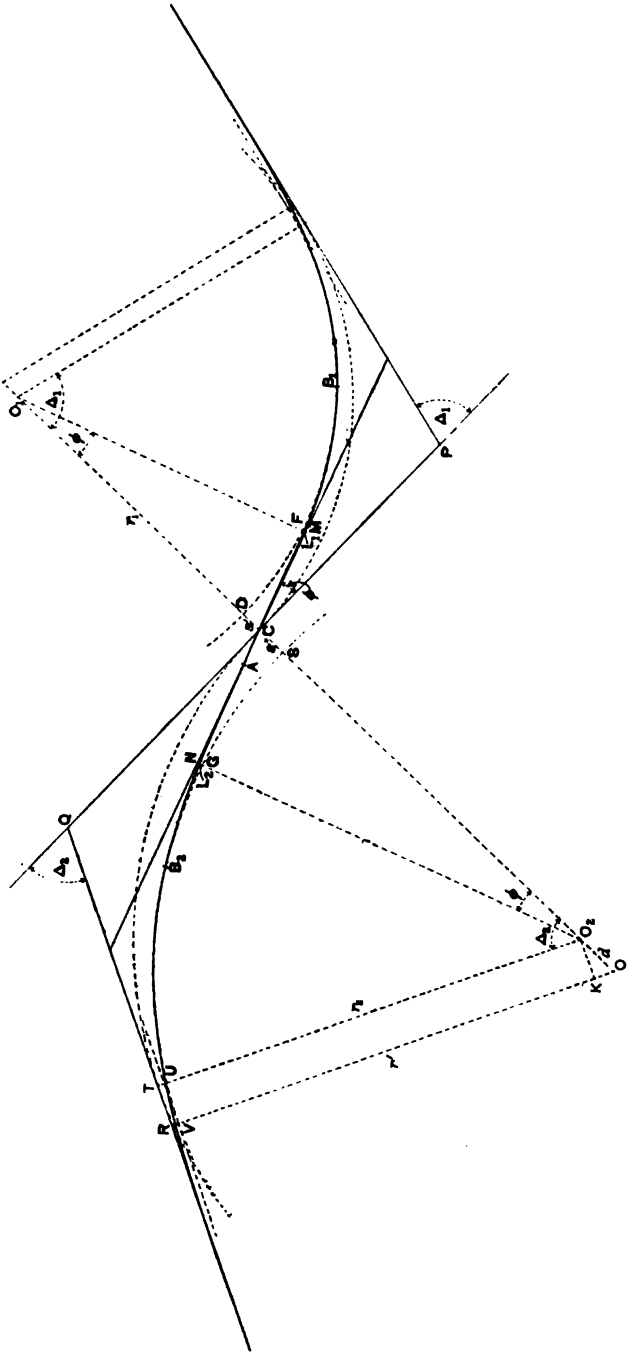

Fig. 51.

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happens to come in the right place, the operation of drawing the circular curve back along the line O_1O_2 of the centres is not sufficient without altering the radius slightly to insure that a spiral from the straight can make a proper connection with it.

Thus it is necessary to draw the line VU parallel to the tangent RT, and at a distance $S_2 = \frac{l_2^2}{24r_2}$ from it. If the new circular curve through S with its centre on line O_1O_2 touches the line at V, VO will be at right angles to tangent VU or RT; now draw O_2K at right angles to this line. Calling O_2O d, the distance the centre is moved in and the central angle at O_2 or O $\triangle_2$, and the new radius r', we have—

$$r_2 = TU + UO_2 = S_2 + r' - d\cos\triangle_2$$
$$r_2 = O_2C = OC - d = r' + S'' - d$$

Equating these values of r_2—

$$r' + s'' - d = S_2 + r' - d\cos\triangle_2$$
$$\therefore s'' - S_2 = d(1 - \cos\triangle_2)$$
$$\therefore d = \frac{s'' - S_2}{1 - \cos\triangle_2}$$

Again $$r' = r_2 - s'' + d = r_2 - s'' + \frac{s'' - S_2}{1 - \cos\triangle_2}$$

and $$RT = O_2K = d\sin\triangle_2 = \frac{s'' - S_2}{1 - \cos\triangle_2}\cdot\sin\triangle_2$$

It is assumed in this example $\triangle_2 = 20°$.

Therefore $d = \frac{3{\cdot}96 - {\cdot}51}{{\cdot}06031} = 57{\cdot}2$ ft., giving the amount the centre is shifted back, and $r' = r_2 + 57{\cdot}2 - 3{\cdot}96 = r_2 + 53{\cdot}24$ ft., which gives the increase in the length of the radius.

The positions of R and V, the extremities of the shift, are determined by measuring from T, the original tangent point, a distance TR or $d\sin\triangle_2 = 57{\cdot}2 \times \sin 20° = 57{\cdot}2 \times {\cdot}342 = 19{\cdot}56$ ft., and setting off an offset $= S_2$ or ·51 ft.

The new radius of curvature $r' = r_2 + 53{\cdot}24$ ft. $= 40{\cdot}806$ chains.

This curve must now be set out from S and check on V in the same manner as shown in the last example. The alteration in the curvature would suggest that B_2 when set out from the tangent NACM would not fall on the circular curve. However, even this considerable change in the curvature, nearly amounting to a chain, only alters the position of the point B_2 by a small fraction of an inch. If it should be felt necessary to correct for this the spiral AL_2B_2 can be set out to suit the new curvature without any appreciable effect being felt on the smooth running on the road. The other extremity of the circular curve r_1 must be dealt with in a similar manner.

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The spiral AL_2B_2 can now be set out by deflection angles from A, the shift NG or S_2 being equal to $\frac{l_2^2}{24r_2}$, and $\alpha = 3\theta = \frac{l_2}{2r_2}$ for point B_2 (see p. 131). (The same spiral can be used at the other end of this curve where the shift is RV.) The spiral AL_1B_1 can be set out from A in a similar manner.

Another Method of Solving this Problem.—A more rational and precise method of dealing with the problem of applying transition curves to connect the tangents with the extremes of the compound circular curves after the latter have been moved in to the extent of their respective shifts, as in the above two examples, is that of compounding the circular curves with others of slightly different radii. (The change in the radii is never great enough to make it necessary to insert a spiral between the curve.)

Thus in Fig. 52 curve of radius r_1 has its centre moved from O_2 to O_2'. This draws point T to new position W

$$\therefore SC = s' = O_2O_2' = TW$$

Now set off $TU = S_1$ at right angles to tangent and draw UVQ parallel to tangent. This is now the line which the new circular curve must touch; it must also touch the curve (r_1) at some point at Y (generally fixed by having YB_2 sufficiently long for easy running). Join YO_2' and let O be centre of required circle (r_2), and V its other point of contact. Let central angles $WO_2'Y$ and $WO_2'S$ be $\triangle$ and $180° - \triangle'$ respectively. Then calling $O_2'O$ d, we have—

$$r_1 = WO_2' = s' \cos \triangle' + S_1 + r_2 + d \cos \triangle$$
$$\text{Again } r_1 = O_2'Y = d + r_2$$

Equating these two values of r_1 we get

$$d + r_2 = s' \cos \triangle' + S_1 + r_2 + d \cos \triangle$$
$$\therefore d(1 - \cos \triangle) = s' \cos \triangle' + S_1$$
$$\therefore d = \frac{s' \cos \triangle' + S_1}{1 - \cos \triangle}$$

All quantities in the right half of equation are known because $\triangle' = 180 - \triangle_2$, and $\triangle$ is the angle subtended by an arc

$$= TYS - YB_2 - B_2S, \text{ which are all known.}$$
$$\text{Then } r_2 = r_1 - d = \text{known}$$
$$\text{Now } VZ = OO_2' \sin \triangle = d \sin \triangle$$
$$\text{Also } UZ = s' \sin \triangle'$$
$$\therefore UV = d \sin \triangle - s' \sin \triangle' = \text{known}$$

$\therefore$ Both tangent points V and Y and the radius (r_2) are known. $\therefore$ Curve can be set out.

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Fig. 52 may be looked upon as representing only one of two compounding curves as shown in the previous examples. A similar operation would require to be done in the case of the curve connecting with this one on the right of the figure.

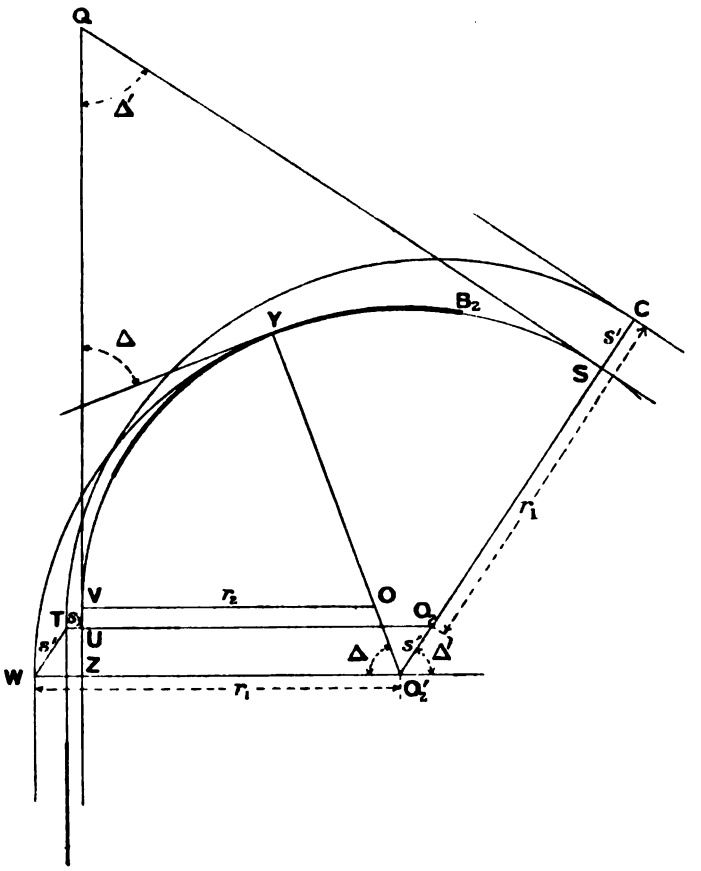

FIG. 52.

REVERSE CURVES CONNECTED BY A COMMON TANGENT

In this case, provided the common tangent is long enough, it is only necessary to deflect it about some point in it (C in Fig. 53) so that the shift for each circle will be the correct one.

If r_1 is the smaller radius, the length of transition l_1 will be $\sqrt{r_1}$. The portion of the same transition for arc (r_2) will be

$$l_2 = \frac{r_1}{r_2}\sqrt{r_1}$$

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Now S_1 (the shift) $= \frac{l_1^2}{24r_1} = \frac{1}{24}$ chains (a constant) and

$$S_2 = \frac{l_2^2}{24r_2} = \frac{1}{24}\frac{r_1^3}{r_2^3} \quad \therefore \frac{S_1}{S_2} = \frac{r_2^3}{r_1^3}$$

In figure $FM = S_1$ and arc of radius $r_1 + S_1$ is described. Similarly $GN = S_2$ and arc of radius $r_2 + S_2$ is described, and common tangent NCM to these two arcs. The angle θ which the two positions of the common tangent make with each other is equal to the angle between the pair of perpendiculars from each centre. If T_1 and T_2 are the sections into which the common tangent is divided by C

The shift $S_1 = PR = T_1 . \theta$ and $S_2 = QS = T_2\theta$. (This is a very close approximation.)

$$\therefore \frac{T_1}{T_2} = \frac{S_1}{S_2} = \frac{r_2^3}{r_1^3}$$

But $T_1 + T_2$ is known. $\therefore T_1$ and T_2 can be found, and $\theta = \frac{S_1}{T_1}$ = known. $\therefore FP = r_1\theta$ = known. Also QG is known. Therefore F and G, the extremities of the shifts, can be found on the

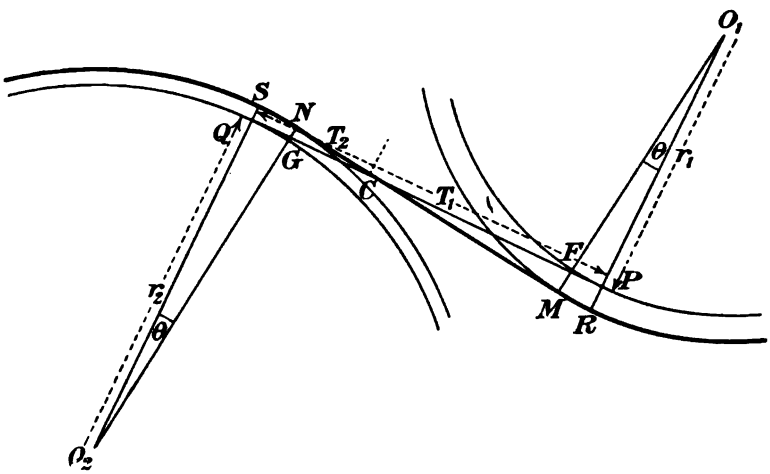

FIG. 53.

ground by measurements from the tangent points P and Q respectively. If FM be made $= S_1$ on the ground, the new tangent line CM can be set out, and when extended to N should give $GN = S_2$.

If from M $\frac{1}{2}l_1$ be set off along MC the spiral tangent point is

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found, and the curve can be run in by deflection angles or offsets. In a similar manner the other spiral can be run in.

If the common tangent is very short, less than $\frac{1}{2}(l_1 + l_2)$, it will be necessary to reduce the lengths of the transitions. This can be done in the same proportion with satisfactory results if the transition is not shortened to less than what will provide 40 feet to the inch of superelevation on the circular arc.

The ban which has been placed on reverse curves in the case of new lines is a wise one, as not alone does the form of such a curve in itself provide serious oscillations in the running over it, but even the superelevation is obliged to be non-existent at the point of reverse where it is most needed. But if a sufficient length of straight can be provided between reverse curves to enable the full-sized transition curves to be inserted, the travelling will be as smooth as could be desired, and thus reverse curves, which so frequently mean a substantial reduction in the cost of construction, can still be used without prejudice.

The formula $l = \sqrt{r}$ for determining the length of transition to be used has much to commend it, though it is used very little on British railways as yet; the custom at present being to take a length of 50, 60 or sometimes 66 feet for every inch of superelevation on the circular curve. It might seem to a permanent way engineer that the length provided by this formula for transitions to sharp curves was much too small, for instead of increasing in length as the curvature becomes sharper it diminishes, consequently making the rate of gain of superelevation much quicker than 1 inch in 60 feet. But as the curves are sharper the maximum speed permitted on them is also less, and thus in surmounting the superelevation at a lower velocity no uncomfortable motion is felt by passengers, as the discomfort of the motion varies as the cube of the velocity. Consequently a speed of 50 miles per hour will cause nearly 5 times as much oscillation as that due to one of 30 miles per hour.

Again, for speeds of 70 and 80 miles per hour and a radius of between 40 and 60 chains, it may seem that the lengths of transitions are much too long, but here the excessive speeds demand a very gentle gradient on which to gain the full cant if the sensation of having the coach heaved up at one side is to be sufficiently gradual. Thus at speeds of 70 to 80 miles per hour the gain of superelevation should amount to 1 inch in 80 feet or more if the best running is to be obtained. Therefore a general rule of, say, 1 inch in 60 feet for the cant on all transition curves is unscientific and not productive of the best results. Yet it is gratifying to know that all our principal railways have now taken to the introduction of transition curves, where possible, at some parts of their systems.

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But the feeling that circular curves of large radius provide sufficiently good running to render them immune from the transition innovation, is one which should be dissipated. It reminds the author of a surveyor taking a reading of an angle with a theodolite, to degrees only, and ignoring the minutes and seconds because it happened to coincide nearly with a certain number of degrees. As long as large easy curves are used for only *moderately high speeds* the need for a transition curve is not great. But the tendency on some of our principal railways to insert curves only of large radius and without transition curves on their main lines is a mistake, as the approach to any of these curves by trains at *high speed* is necessarily accompanied by irregularities of motion which would not occur had it been made on a transition curve.

Also, the difficulty of introducing a spiral in these cases of curves of large radius is by no means great, as the "shift" in the worst cases is only a few inches. Indeed, speaking broadly, it might be said that it is only on curves of large radius that there is any great need for a transition approach, as these curves are generally associated with high speeds, whereas the sharper curves are excluded from lines intended for fast traffic. Also, it must be remembered that on a high-speed track an irregularity so slight as to escape the eye of the platelayer entirely may cause a very serious jar in the running at that point.

There is no doubt a certain amount of easing of the curvature at the tangents of curves of large radius is always performed by the platelayers, but this is of necessity a variable quantity.

This custom among platelayers of making the line "eyeable" is frequently condemned on the grounds that the curve is sharpened by this operation and consequently causes inferior running. In reality the sharpening of the curve is seldom if ever the real cause of the trouble, which is due to sluing the road into a form unsuitable as a transition curve.

Personally the authors have always considered a good platelayer a very worthy and useful servant of a railway, but they could never see any justification for the permanent way engineer handing over to him the duty of adjusting the most difficult parts of his track. Some of our railway engineers have now realized the seriousness of adopting such a custom, and in retaking the reins of control into their own hands have naturally found it necessary to adopt a very firm attitude towards platelayers in regard to these matters, demanding that instructions from the office were alone to be followed and that these were to be carried out with minute accuracy.

It is advisable that marks of a permanent nature be fixed along the 6-foot way from which measurements may be taken to the curve in order to re-fix it in its true position and to the correct elevation

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during the operation of re-ballasting or other renewal work. The foreman platelayer should be provided with these and a plan giving the lengths of the offsets. A type of mark which has been adopted with some success is a centre peg made of a fish-plate set in a block of concrete, the fish-plate having a saw-cut in it to indicate the centre line of the railway. It is not possible to fix these at the level of the rails as their projection above the surface of the 6-foot way would be a danger to platelayers, also if a material train was unloaded at such a point a rail dropped on the marks would destroy their accuracy. However, if fixed well down in the pitching and formation so that the top surface of the plate is flush with the top surface of the sleeper or at a definite number of inches, say, 7 below the surface of the rail, they can be always used for re-establishing the rail in its correct position.

Another method is that of using short lengths of old flatfoot rails with the flange turned towards the line and placed so that the ordinary road gauge measured the distance from the outside of the running rail head to the flange of the rail used as a mark. These pegs were put in at intervals of 25 feet, either on the inside or outside of the curve (the line was single), as the ground was more suitable and each was set so that its top was at the correct level of the corresponding rail at the point and the correct superelevation was marked on it with a punch.

But to return to the matter of spiralling flat and sharp curves, the custom of making the length of the transition proportional to the number of inches of superelevation was adopted in America probably for the purpose of obtaining easy hauling for freight trains travelling at a moderate speed. The Canadian Pacific Railway, as also some of the leading railways in this country, adopts 60 feet for every inch of superelevation on the circular curve, but the former seldom uses more than 5 inches of superelevation. Also, spirals are only used for curves of superelevation of 3 inches and upwards. The tendency in America is to keep down the superelevation as much as possible. This is for the benefit of freight trains, which are considered of primary importance but travel at lower speeds, as the higher superelevation means increased resistance to hauling, caused by the wheels binding against the lower rail. The American Railway Engineering Association has laid down the rule that in all cases of 2 inches of superelevation and upwards transition curves should be inserted This obviously is intended to provide for speeds much in excess of those attained on the Canadian Pacific Railway.

Objection has often been raised to the method of inserting a transition curve by sharpening the curvature of the circular curve where it is not possible to move it to form the shift, the objection

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being that in order to effect a better approach the curve has been rendered more dangerous by having its radius reduced. This argument on the face of it seems to carry some weight. But the principal point of danger of a sharp curve is at its entry. If trains could be brought on to curves without any sudden irregularity of motion before settling down to the uniform motion on the curve, very much sharper curves could be adopted for high-speed trains than are used to-day. Indeed the results of experience and experiments seem to show that trains traverse curves of 5 chains radius at 35 miles per hour with perfect ease and without showing any tendency to leave the metals.

However, the principal cases here referred to occurred on the Ceylon Government and South African railways, but if tried on English railways would, no doubt, require the track to be laid wide to gauge, as curves of 5 or sometimes 6 chains are the sharpest adopted anywhere on English railways. Still the fact remains that trains can traverse such curves at 35 miles per hour. But the curve ordinarily provided for such a speed would be one of about 15 chains radius and its reduction to one of 13 chains radius would not seriously affect the regularity of motion in passing round it at 35 miles per hour. However, this is on the presumption that no dangerous oscillations are set up at its approach to the curve. This reduction of the radius of curvature for a few chain lengths of the curve will provide an ample shift to enable the most efficient transition curve to be inserted. Accordingly, the motion from the straight to the 13 chain curve will be quite uniform, and also that from the 13 chain to the 15 chain radius curve, being a change of less than 15 per cent.

There is no hard and fast rule adopted in this country with regard to the number of points set out on the ground to fix each transition curve, the tendency being to fix 8 or even 10 points on a long transition curve and 6 on the shorter ones. A point at every half chain is generally sufficient, but some prefer to fix points about 1 chain apart at the tangent end and half that amount at the circular curve end. This gives good results, as the curvature is so much sharper at that end, and wants its position defining at nearer points.

On the Canadian Pacific Railway it is usual to set out the centre point and two-quarter points only in addition to the extremities, except in the case of very long transition curves. If these are set out by deflection angles from the beginning of the spiral they are in the ratio of 1 : 4 : 9 : 16, as they vary as the square of the length. Also if set out from the end of the spiral the ratio of the four angles is 11 : 20 : 27 : 32. So when any one of these angles is found (generally that to the end point) the others are easily calculated.

The accuracy of the curvature on spirals existing on the ground can be tested by the chord and versine method in a similar manner

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to testing the curvature of an existing circular curve. The methods of calculating these versines on transition curves for different cases are given in Chapter IX.

To plot curves from versines to greatly distorted scales in order to magnify the irregularities of curvature and enable corrections to be worked out on an office plan, a method is given at length in Chapter IX.

It will be interesting, however, to many to know that this method has been adopted to a greater or less degree on nearly every railway in the country. But it was found in some cases that the exceptionally good draughtsmanship required, even with the distorted scales generally used, rendered the operation difficult to perform with the amount of precision desired, the improvements to the existing rails, often amounting at several points to shifting it small fractions of an inch, depended for its accuracy upon measurements scaled upon the plan, and very small errors in the measurements or in the draughtsmanship were liable to cause considerable errors in the curvature, as can be easily understood by anyone accustomed to measure curvatures by the versine method on the ground. Accordingly, a co-ordinate method of plotting the results on squared paper has been tried with very satisfactory results, as any distortion due to expansion or contraction of the paper or poor draughtsmanship does not affect the results. For particulars of these methods see Chapter IX.

The suggestions for improvements in the railway lines of this country, as here set out, are not intended to place too adverse a criticism on the efficiency of the permanent way work in the past, as the ease and comfort with which we traverse long stretches of some of our main lines at very high speeds has given our railways a great reputation all over the world.

Much of this easy running is due probably to the adoption of long lengths of curves of large radius, it having been realized that the motion of a train on a curve of large radius was better than on any straight, because there was less oscillation, practically no extra resistance and a slight bearing on the outer rail, which seemed to steady the train when it travelled at a high velocity.

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CHAPTER IX

THE SPIRALLING OF EXISTING RAILWAY CURVES

THE advantages of the transition curve towards bringing about easy running, have become so familiar to railway engineers that not alone is the spiral being used on all new roads of any importance, but decided efforts are being made even on English railways to improve the existing curves by some such means, where it is possible to do so. The improvements which have been effected by even slightly sluing existing roads to conform more with a spiral, sometimes only a few inches, have been so remarkable, that the properties of transition curves are now being more thoroughly investigated. In the case of existing roads, in order to investigate thoroughly each individual case it is necessary to make an accurate survey of the existing rails, also showing the outside of the formations on both sides of the line at each chain point and the position of all fixed points such as bridge abutments and piers or any object liable to foul the construction-gauge. It is also advisable to record in the field notes the depth of cutting or embankment at each chain point in the event of its being necessary to widen the formation. But even when curves like this are plotted to as large a scale as 1 inch = 8 feet it is difficult to recognize any irregularities in the curvature. Also, although existing curves are nominally of so many chains radius, when tested by centre-offsets at the centre of a constant length of chord for each chain point round a curve, they are often found to vary as much as 8 or 10 per cent. from the normal in chains radius, between one chain and the next. This variation in curvature on an existing railway curve is not apparent to the eye when examining it on the ground and is evidently caused by the platelayers when packing the road at ballasting times. The sluing from the true position is caused by the platelayers when the foreman is "eyeing in" the road to what he thinks correct, subsequently. Pegs are always driven by the engineer, every chain length, into the formation from which to measure for the rails. But a new road when being put in requires much sluing from side to side before its appearance satisfies the foreman platelayer,

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and it is questionable whether he always adheres implicitly to the peg measurements. The keen permanent-way engineer will early impress upon his outside staff that instructions supplied to them by his office, either in the form of pegs or written directions, must be carried out accurately and to the letter.

Method of Magnifying the Curvature in Plan.—But in dealing with the survey of an existing curve in the office, even when plotted to a large scale, much difficulty has been felt in recognizing irregularities in curvatures, which, though small at times, are very important when the question of curve improvements has to be dealt with. In a paper given by Mr. W. H. Shortt, A.M.Inst.C.E., entitled "A practical method for the improvement of existing railway curves," in 1909, before the Institution of Civil Engineers, the author suggested an excellent method of magnifying the curvature in plan. This was effected in the following way :—

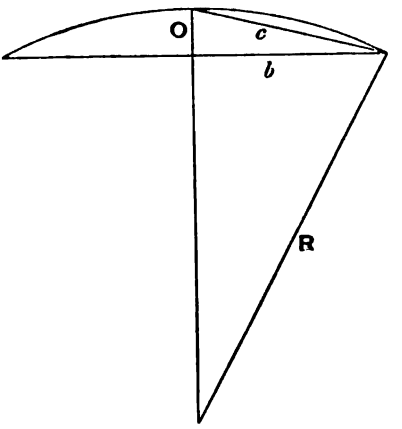

FIG. 54.

All lengths measured along the outer rail which was used as a base line, were plotted to a scale of 40 feet to an inch ; all lengths at right angles to this base line to a scale of 4 feet to an inch, and all radii to a scale of 400 feet to an inch. These scales are inter-dependent, the fixing of any two of them determines the third. Fig. 54 shows that in a circular curve $O \times (2R - O) = b^2$.

$$\therefore O \times 2R = b^2 + O^2 = c^2$$

We may look upon c as representing measurements taken along the curve, as they are always chord lengths, O as measurements at right angles to c, and R as radii measurements. Now, the equation $O \times 2R = c^2$ gives the property of a circular curve only. Accordingly if O, R and c are lengths in feet, the scales to which each of these is plotted must enable that property to be preserved as well after plotting as before.

Thus, in the case of the above-mentioned scales we have the lengths of O, R, and c, in inches for plotting purposes as follows :—

$$\frac{O}{4}, \quad \frac{R}{400} \text{ and } \frac{c}{40}$$

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and if $O \times 2R = c^2$ to begin with, even when the parts are plotted to these scales, it will continue to do so for

$$\frac{O}{4} \times \frac{2R}{400} = \frac{c^2}{(40)^2}$$

Since $4 \times 400 = 40 \times 40$

Again, other scales can be used for the purpose, as, for instance, 2, 200 and 20 feet to an inch respectively. This latter has been found a very convenient group of scales for this work by permanent-way engineers. Now it is necessary to remember, when considering the correctness of this method, that the plotting of the curve is only performed by using chord offsets and therefore that each short portion of the curve is added to that previously plotted to form the whole curve. Thus Fig. 55 shows a portion of existing curve leading off from the straight plotted to this distorted scale, the actual measurements on the ground being taken to the outer edge of the outer rail of the curve. On this the chain points are numbered. The offsets to determine the curvature were taken at the centre of a 2-chain chord stretched against the rail, these being taken at each chain point (see Table from Field Notes, p. 168). Whilst these notes are being taken on the ground, notes are also made of the distances from the outer rail to the outside of the formation at each chain point and to fix the positions of bridges, signal posts, cabins, etc.

Plotting the Curve to the Distorted Scale.—The plotting of the curve to the distorted scale is effected as follows:—Starting from chainage 0. The 2-chain chord gives an offset of 0 at 1[1], and again an offset of 0 at 2, indicating that the line is straight from 0 to 3. At 3 the offset is $1\frac{1}{2}$ inches, indicating that the rail curved away 3 inches from the straight at 4. This enables 4 to be plotted. The method usually adopted for plotting points with such small offsets, is to extend the chain line to four or five times its length, and draw the offset magnified by the same amount, and join its extremity to the chain point. Thus in Fig 55, 03 has been extended to 3_1 four times the length from 3 to 4, and the offset set up at 3_1 has been 12 inches instead of 3. Then setting off along the joining line one chain gives point 4. Again, since the offset at 4 is $4\frac{1}{2}$ inches, the point 5 lies 9 inches to the left of the line joining 3 and 4. Thus, as before, extending line 3, 4, to four times its length, and setting off an offset of 36 inches, and joining the end of this to point 4, and measuring off one chain length along this from 4 gives point 5. In this way the whole curve can be plotted. But this method is tedious, and is generally necessary for the first few points only, where the curvature is flat. The subsequent points are on a curve of the full sharpness of the

[1] See Table XII., p. 168.

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circular curve, and will be capable of being found by finding the centre of curvature of each 2-chain lengths of the curve, provided that curvature is sufficiently sharp to enable the centre to come on the paper and be plotted by means of a beam compass. From a table of versed sines the radii of curvature for each 2-chain lengths may be found. If the table is only prepared for chords of 1-chain length, the central offsets from the 2-chain chord may be reduced to the corresponding ones for a 1-chain chord by dividing them by 4. Thus a 2-chain chord central offset of 6 inches belongs to a circle whose radius is the same as a 1-chain chord central offset of $1\frac{1}{2}$ inches.

Thus points 4 and 5, having been found by the first method, and the radius of curvature of the 2 chains from 4 to 6 having been found from the table of versed sines for the offset at 5, the centre can be found with a compass by striking intersecting arcs with this radius from 4 and 5. This being the centre for 6, point 6 may be plotted by striking an arc with this radius from the centre point just found, and an arc of radius 1 chain from point 5 to intersect it. For example, the offset at 5 is $5\frac{3}{4}$ inches, which gives a radius of curvature of 68·9 chains. Taking 4 and 5 successively as centres, with a radius of 68·9 chains, plot the arcs which intersect at X in Fig. 55, X is therefore the centre for point 6, which is 68·9 chains from X, and 1 chain from 5. Again, the offset at 6 is $11\frac{7}{8}$ inches, which gives 33·3 chains as the radius for the circle passing through points 5, 6, and 7. The new centre Y is fixed from points 5 and 6 in the same way as X was from points 4 and 5, and so on. When the chain points are plotted thus a smooth curve is drawn through them all. Thus the curvature, and its variations, being much exaggerated, the portions requiring improving become more apparent to the eye.

Using the Distorted Plan to Effect Improvements.—When this plan of the existing road is complete, the problem of improving it can be dealt with. The sharpest portion of the curve, if little sharper than the average or original curvature, will probably determine the basis for improvement. Improvements in existing curves can generally be effected only by sharpening a portion of the curve in order to enable the gradual transition to be inserted. For it is very natural to expect that if a slight sluing of an existing road flattens one portion of it, that portion immediately following becomes sharper. The sharpest portion of curve, with one exception, being of 32·3 chains radius. The method of taking the square root of this to determine the length of the transition curve gives 5·6 chains for the latter. It is advisable to plot this transition curve on tracing paper.

In Fig. 55A a convenient curvature of 32·6 was taken, and the length of the transition curve 5·7 chains. Here the amount of shift is 2·745 feet, or practically 2′ 9″, and the calculated offsets for five equal

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intervals from either end of the curve being as follows: ·13, 1·04, 3·53, 8·44, and 16·50 inches.

Table XII., below, shows the central chord offsets at each chain point on the curve, also the corresponding radii in chains. It also gives the limits to which the line can be slued, either to the right or left, dependent upon the width of formation at each point. When this plan is to be used in the office, and is consequently plotted to a large size, these available limits to which the road can be slued are plotted on the plan. These have been omitted in Fig. 55 (with the exception of two, where the limits are small and very binding) to save confusion. It should be noted that the widths of formation, and such tying points as bridge abutments, and other permanent structures are not plotted in their correct positions, but are placed at such a distance on the right and left of the distorted curve-line at each point, as is given by the limits of sluing at the point.

TABLE XII.

TABLE FROM FIELD NOTES.

Chain point	Two-chain chord offset in inches.	Corresponding radii in chains.	Maximum width available for sluing line.		
			On inside of curve.	On outside of curve.	
0 + 00	0	∞	1′ 8″	2′ 8″	
1 + 00	0	∞	1′ 4″	2′ 8′	
2 + 00	0	∞	1′ 10″	2′ 6″	
3 + 00	1½	264	2′ 0″	2′ 6″	
4 + 00	4½	88	2′ 0″	2′ 4″	
5 + 00	5¾	68·9	1′ 6″	2′ 2″	
6 + 00	11⅞	33·3	1′ 4″	2′ 0″	
			10″	1′ 4″	Bridge abutments.
7 + 00	12	33	1′ 6″	1′ 6″	
8 + 00	12	33	1′ 10″	1′ 2″	
9 + 00	12	33	2′ 2″	8″	
10 + 00	11⅞	33·3	2′ 0″	1′ 0″	
11 + 00	11¾	33·7	1′ 10″	4′	Bridge abutments.
12 + 00	12¼	32·3	2′ 6″	10″	
13 + 00	12¼	32·3	2′ 8″	1′ 4″	
14 + 00	12⅛	31·7	3′ 0″	1′ 4″	
15 + 00	11¾	33·7	3′ 2″	1′ 10″	
16 + 00	12	33	3′ 6″	2′ 0″	
17 + 00	12¼	32·3	3′ 6″	2′ 2″	
18 + 00	11½	34·4	3′ 4″	2′ 0″	
19 + 00	11⅝	34	3′ 0″	2′ 0″	
20 + 00	11⅞	33·3	2′ 9″	1′ 10″	

As soon as the spiral and trial circular curve are plotted on tracing paper (see Fig. 55A) this is placed over the plan of the existing road,

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and moved about until a suitable position is found, if possible, to which the road could be slued with advantage. It will be observed, when trying to place the spiral and circular curve of 32·6 chain radius, shown in full lines in Fig. 55A in such a position that this is impossible,

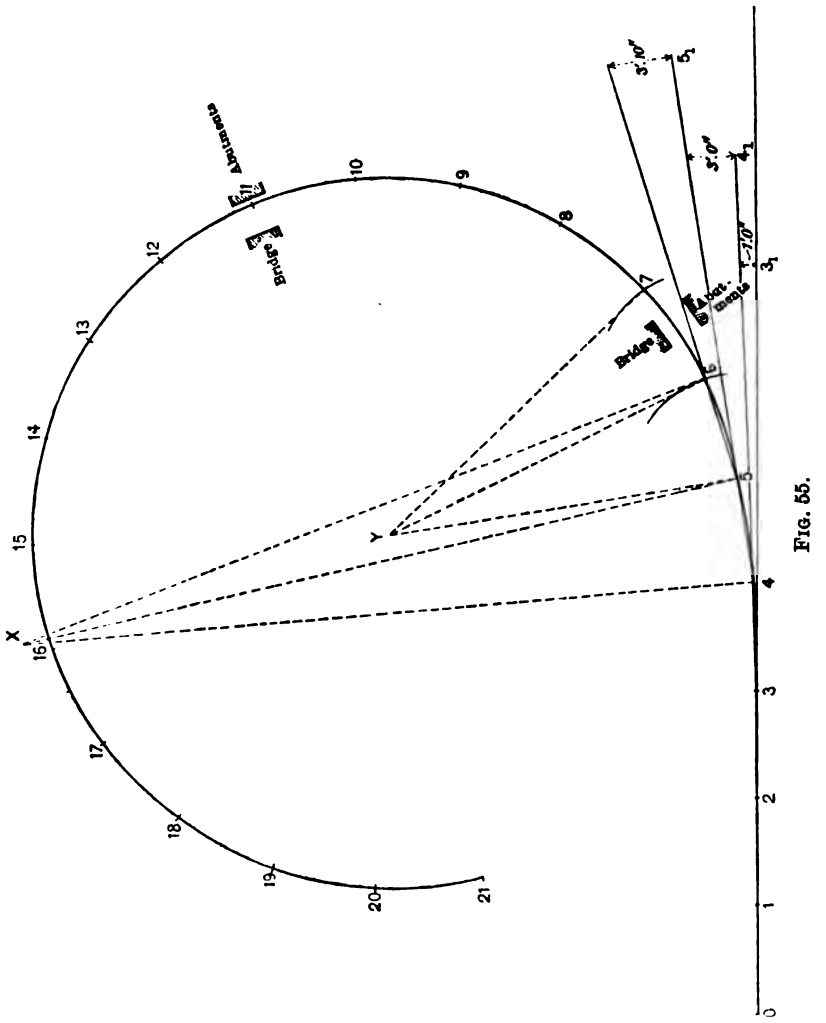

Fig. 55.

because the curve, when just clearing the abutment on the outside of the curve at chain point 11 + 00, fouls the abutment on the inside of the curve at 6 + 60 chainage. Of course, in applying these plans to one another, it is essential to keep the tangent lines coincident.

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Accordingly, as it was evident that the 32·6 chain curve was not sharp enough, a 31·75 chain curve was constructed with the same shift, as shown dotted in Fig. 55A. The spiral for this new curve is not shown, being almost coincident with the first spiral except close to the E.S.,

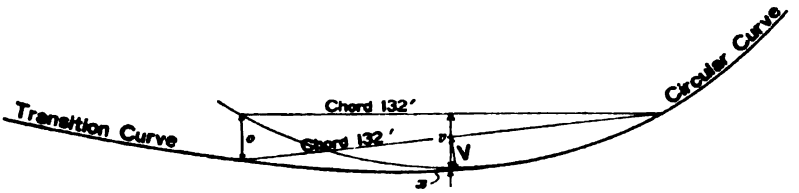

FIG. 56.

This has the sharpest curvature to be found in the existing curve, which occurs at chain point 14 + 00. When this curve was applied to plan (Fig. 55) it was found by sliding the B.S. along the tangent to a point at about 1·8 chainage, very good results could be obtained. The bridge

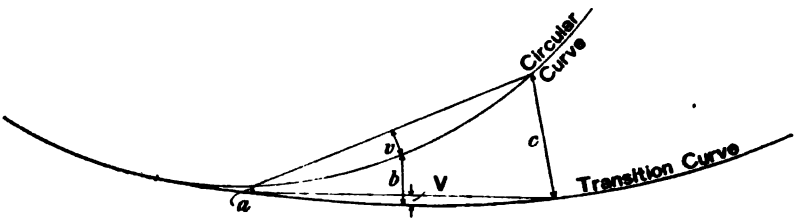

FIG. 57.

abutment on the inside of the curve at 6 + 60 was not fouled, and at 11 + 00 the other tight point, the curve almost coincided with that of the existing road. It was also evident that no sluing would be necessary beyond chain point 12 + 00. In shifting the road to the position given by this curve, it was found that all sluing necessary was towards the inside of the curve, the amount at the various points being as follows :—

Chain point	3 + 00	$\frac{3}{4}$	inches
,, ,,	4 + 00	4	,,
,, ,,	5 + 00	$9\frac{1}{2}$	,,
,, ,,	6 + 00	$10\frac{1}{4}$	,,
,, ,,	7 + 00	10	,,
,, ,,	8 + 00	$7\frac{1}{2}$	,,
,, ,,	9 + 00	$2\frac{1}{2}$	,,
,, ,,	10 + 00	1	,,
,, ,,	11 + 00	$\frac{1}{4}$	,,
,, ,,	12 + 00	0	,,

All these are within the limits permissible according to the Table from Field Notes.

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Thus it can be seen the sharpest curvature found in the existing curve is more likely to give the best results when spiralling an existing road. But it may sometimes be necessary to use even a sharper curve. This is generally necessary in the case of a reverse curve, where there is little or no straight between the curves. In such a case by connecting two sharper curves by two transition curves, a solution to the difficulty may be found by varying the lengths of the transition curves, and if necessary, the curvature of the trial circular curves. By adopting these methods, wonderful improvements have been effected in the running on many existing roads in Great Britain, and in many places where coaches and trucks were liable to leave the metals, the tendency entirely disappeared.

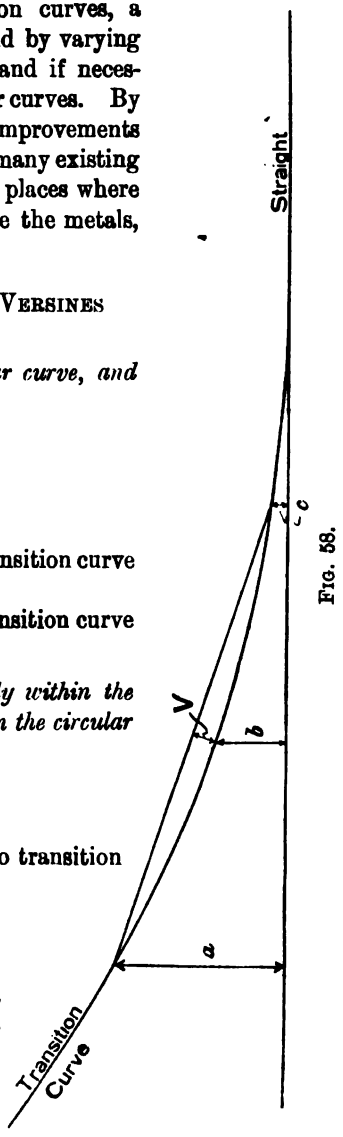

FIG. 58.

USEFUL FORMULÆ WHEN USING VERSINES ON TRANSITION CURVES

Case 1.—*Chord partly on circular curve, and partly on transition* (Fig. 56).

$$V = v - \frac{o}{2} + x$$

V = Versine on transition curve.
v = Versine on circular curve.
o = Offset from circular curve to transition curve at extremity of chord.
x = Offset from circular curve to transition curve at centre of chord.

Case 2.—*Chord of* 132′ *lies wholly within the transition, and the offsets are taken from the circular curve produced* (Fig. 57).

v = Versine on circular curve.
V = Versine on transition curve.
a, b, c = Distance from circular curve to transition curve.

$$V = v - \frac{a + c}{2} + b$$

Case 3.—*Chord of* 132′ *lies wholly within the transition curve, and the offsets are taken from the straight produced* (Fig. 58).

$$V = \frac{a + c}{2} - b$$

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Case 4.—*Chord lies partly on one circular curve, and partly on another* (Fig. 59).

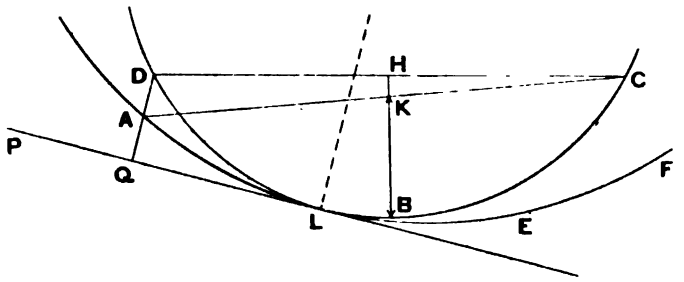

Fig. 59.

Let chord AC stretch from point C on curve CBLD to point A on curve FELA, and let KB be the required versine:

Let L be the common tangent point to both curves.

Through L draw PL tangential to both curves.

Then QA and QD are offsets to curves CBLD and FELA at distance LQ from tangent point.

Then KB = HB (versine for curve CBLD) minus $\frac{1}{2}$ (QA − QD).

Note the minus sign − because FELA is a flatter curve than CBLD.

Useful Formulæ for Everyday Use

In applying the methods set out in this chapter, the following two cases occur frequently in everyday practice on some of the British railways:—

I. **Transition Curve connecting a Straight with a 20-Chain Radius Curve.**—(See Fig. 60.)

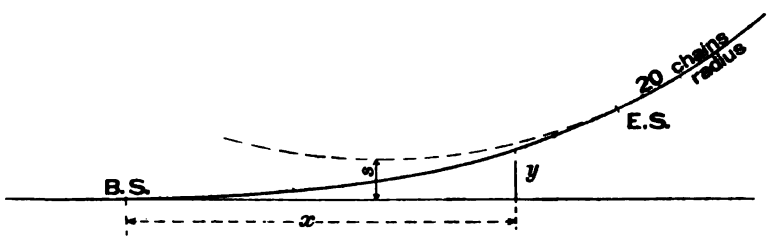

Fig. 60.

Assuming the superelevation to be gained by 1″ in 50′ or 1 in 600, and the amount of it for a 20-chain curve to be ·6′

The length of transition curve L = ·6 × 600 = 360 feet.

Then S (the shift) = $\frac{L^2}{24R} = \frac{360^2}{24 \times 20 \times 66} = 4{\cdot}09$ feet.

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y (the offset at any distance x from the B.S.) is given by :—

$$y = \frac{x^3}{L^3} \cdot 4S$$

$\therefore$ at middle of transition curve $y = \frac{S}{2}$

(Offsets to be taken at right angles to the tangent and length measured along the transition curve.)

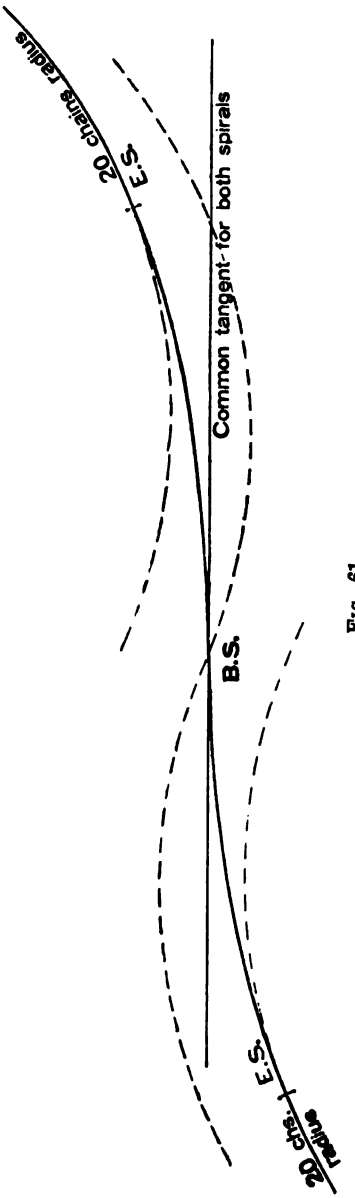

Fig. 61.

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II. **Transition Curves connecting two opposite 20-Chain Radius Curves.**—(See Fig. 61.)

Here the superelevation on each curve = ·6′, therefore the length of each spiral from B.S. to E.S. is 360 feet.

The distance D between 20-chain curves at their nearest point B.S. = 8S. (∵ for each spiral the offset to the circular curve from the common tangent is 4S.)

But S = 4·09 feet (see Case 1).

∴ D = 8S = 32·73 feet.

This is very conveniently given by the formula—

$$D = \frac{L^2}{24} \cdot 4\left(\frac{1}{R} +^{*} \frac{1}{R}\right)$$

[* The plus sign being used when the curves are opposite and the minus sign when in the same direction.]

In this case R and R^1 are equal.

Also $y = \frac{x^3}{L^3} \cdot 4S$ (as in Case 1) measured from the B.S.

A convenient formula for setting out these spiral points by deflection angles is the following:—Deflection angle in minutes to any chord point is given by $\frac{N^2L}{10}$, where N = number of chords between point where instrument is set and the required point, L = length of chords in feet.

For example, in setting out a spiral for a 4° curve. Assuming superelevation to be 4 inches, the length of spiral will be 240 feet, and the lengths of each of the four chords will be 60 feet.

Thus the instrument being set at the B.S. the deflection angle in minutes from the tangent to the third chord point (or $\frac{3}{4}$ pt. of spiral)

$$= \frac{N^2L}{10} = \frac{3^260}{10} = 54'$$

This formula will not apply if the instrument is set up at the intermediate chord points.

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CHAPTER X

RAILWAY SURVEYING IN THE UNITED KINGDOM

Ordnance Survey Maps

The United Kingdom has been completely surveyed and mapped by the Ordnance Surveys and a 6-inches to the mile map of any part of it can be obtained. Also 25-inch maps can be obtained of many parts of the country, more especially of thickly inhabited districts. These maps are full of topographical detail and the 6-inch maps show contours beginning at 50, then 100, 200 and intervals of 100 feet above ordnance datum. For parts of the North of England and South of Scotland the 6-inch maps show interpolated contours at 25-feet intervals.

All 6-inch maps show ordnance bench-marks to one-tenth of a foot.

The 25-inch (or 25·344 inches to the mile map) does not show contours, but all bench-marks, also frequent spot levels on the surface of many main roads are shown.

On account of the existence of these maps and the difficulties imposed upon railway projects by property owners, the expense of acquiring land and the frequent governing points or the need of making the line pass near numerous towns and villages, the same attention is not paid to the contours to obtain the most economical line, from the construction point of view, as in little developed and uninhabited countries.

Preliminary and Parliamentary Plans.—Thus it is that the reconnaissance and preliminary surveys in the ordinary sense are almost entirely omitted. The 6-inch ordnance map is accepted as the first preliminary plan and on it the first trial line is projected. This is also convenient as the scale is suitable for Parliamentary plans, which must be first prepared with sections and estimates and submitted to Parliament in order to obtain the sanction necessary before proceeding with the construction of the line. The bench-marks and spot levels upon the public roads (obtained from the 25-inch map) are of great help in locating this first line, but the contours being very far apart (100-feet

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vertical intervals) a thorough and exhaustive location cannot be effected.

The office work necessary for laying down the first location line is as follows:—

First obtain the ordnance sheets showing the area through which the line must pass, and cut them in strips about 12 to 15 inches wide and mount them on linen or paste them together. It is advisable to make sure that the strip is wide enough and the centre line kept, at all points, at least 4 inches from the edge of the plan, especially if the line is liable to curve about much.

Location

The Trial Location in the Office.—The governing points are now studied and the approximate surface levels at them obtained. A trial line may now be sketched in between these, by observing the gain or loss in height to be obtained in passing from one to the next. For example, if a point B is 6 inches (or 1 mile) from A and 50 feet higher than it, and if it be assumed that A is on the 200-foot contour, five intermediate points can be located about 1 inch apart between these points starting from A. Each of these points can be fixed respectively upon the 208, 216, 225, 233, 242 contours, interpolated between the 200 and 300, and a dotted line sketched in pencil through these points and A and B. The interpolation need only be done roughly, as it assumes a uniform slope between the 300 and 200 contours which probably does not exist. Similarly between the governing points B and C, C and D, etc., the dotted line can be extended, each one being the approximate surface line for the gradient between the points until a long length of dotted line is thus pencilled on. Now this dotted line winds about in part depending upon the forms of the contour lines. But the located line must be composed of lengths of straight and easy curves; consequently, it is only possible to draw a system of straights and curves which will follow the dotted line as nearly as possible. In this way a trial line is plotted on the plan which will probably indicate light earthwork if it follows the dotted line closely and heavy earthwork if it frequently departs from it considerably. But as the contours have only been interpolated this cannot be relied upon implicitly.

Checking the Office Location in the Field.—The plan should now be taken into the field and the line marked out on it walked over to check the assumed topography by comparison with the real topography. But before leaving the office the dotted line should have marked upon it the surface gradients at the different parts which it attempts to follow. A hand-level will be found useful in occasionally checking the contours. When on the ground the form of the surface will suggest whether interpo-

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lation was sufficiently accurate in sketching parts of contour lines. For instance, if both the 200 and 300 contour lines on the plan curved in the same direction for a certain distance the intermediate contours will follow them at uniform intervals, if the ground between the contours slopes evenly the interpolated lines will be about correct. But if the surface between the contours rises in a convex curve the apex of which is above the trial line the important interpolated contours are too far apart, the amount of which can be estimated on the ground and the dotted line corrected accordingly. Again, if the surface between the contours was concave above the trial line, the important interpolated contours are too close together and a corresponding correction requires to be made. At all points where reduced levels are available the level of the trial line should be checked and also the interpolated contours near the point. These testing operations must be started carefully and some time spent over the first mile or two in getting familiar with the method of checking, and rough checks upon the degrees of accuracy obtained should be made, then the subsequent parts of the line will be checked more easily and quickly and the information obtained will be felt to be reliable. The governing points must be carefully investigated and the actual level of the trial line at this point obtained. This can be done sufficiently accurately with the hand-level if a reduced level can be obtained within 300 or 400 feet. The positions of a few 5 or 10-foot contour points on each side of the trial line, at the governing point, can then be marked on with advantage. If the governing point is at a summit or point of highest level, any lower position for the trial line at this point which appears feasible should be sketched on the plan. Throughout this operation of checking the line positions are only determined on the plan by their distances from corners of fences and other prominent objects on the ground which are shown on the ordnance plan. This is sufficiently correct for the purpose on account of the accuracy of the ordnance maps.

If, as frequently occurs, in these operations the ordnance bench-marks shown on the plan are in many cases at a greater distance from the located line than about 600 or 700 feet, it is advisable to employ one or two levelmen. These levelmen, with surveyor's levels, and their staffmen, will start alternately at each of the more remote bench-marks and carry the reduced levels near to the located line, so that the checking surveyor advancing along the located line may pick up these levels with his hand-level and thus more accurately check his contours. In the practice of this method it is generally convenient for the levelman to wait near the line until the surveyor takes a reading on the collimation line of the telescope or on the staff resting on a spot the reduced level of which was determined. The levelman, having thus provided a near bench-mark, takes up his instrument and walks to the next ordnance

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bench-mark from which he is to start. Thus the surveyor in going forward finds sufficient bench-marks provided for him to enable his contour lines to be sketched in with considerable accuracy and much ground covered in a short space of time. There is special need for a fair degree of accuracy in the case of flat or gently sloping country, because there a change of a foot in level might mean altering the position of the line laterally some hundreds of feet.

Parliamentary Plan Limits.—In this way the need of a topographical plan showing contours every 5 feet may be temporarily obviated, and the position of the located line fixed sufficiently accurately for the Parliamentary survey which allows lateral freedom of 300 feet on each side of the line in country parts, and 30 feet in towns. It also allows a change of level of not more than 5 feet in the country, and 2 feet in the town, from that sanctioned by Parliament.

Correcting the Office Location after Checking in the Field.—The additional data now obtained on the site in walking over the proposed line is taken to the office, and the dotted line corrected throughout its length. Also, on account of fresh information about some governing points, new surface gradient lines will require to be dotted in pencil to take the place of those already shown, and on the plan thus modified a new line must be located following the dotted line as nearly as straight and limiting curves will permit. The effect of this line on existing works must be studied. All road crossings must be examined to see if an under or overbridge will be required and consideration given to the effect the necessary headway will have on the level of the railway, or that of the existing road, or both. Thus the necessary headway in crossing public roads should be allowed for in fixing the location line at this stage. It will frequently be found that this will seriously increase the steepness of gradients and also the quantity of earthwork on the trial location, but as this location is about to be modified these objectionable conditions can be in part remedied.

Ruling Gradient.—Sufficient information should by now be obtained to fix the ruling gradient of the portion of line under consideration. This may have been already fixed, as the portion of line to be constructed may be an intended extension of a section of line already in operation, the new part to be operated under the same conditions. In this case, unless what appeared to be the easiest ruling gradient obtainable was steeper than that required, the line would be located and the necessary Parliamentary plans prepared. If, however, the estimated railway gradient was somewhat steeper than that demanded, then a further trial location should be made, and a longitudinal section prepared from the plan. On this latter an attempt

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should be made to locate a gradient line which at no place exceed in steepness that of the stipulated ruling gradient. This may be possible by increasing the earthwork and be perhaps improved by a further visit to the site ; but in some cases the attempts to reduce a natural ruling gradient would only be accompanied by a prohibitive cost of construction.

Maximum Curvature.—Maximum curvature must next be considered. If this has been stipulated and the country is fairly easy for railway work, little difficulty should be experienced in keeping the curves up to the standard required, perhaps with a little additional cost of construction at occasional points. But if the country is rugged and mountainous, the peculiar natural forms of the bases of the hills round which the line skirts will dictate what curves must be used and seldom will it be possible to modify it sufficiently to make the attempt worth trying.

Curvature, especially sharp curvature, must be avoided on ruling gradients, but if some curvature must exist on such gradients, compensation for it should be made by reducing the steepness of the gradient throughout the length of the curve.

Locating the Line after Revision.—Now, having decided upon the ruling gradient and the maximum curvature to be used, the next operation is to locate the revised line on the plan.

When the new dotted line corresponding to the gradients required for a line which will cross all roads at the levels of required headway and also all other critical points, has been completed, a system of straights may be drawn in, by using a straight-edge, to follow the dotted line as nearly as possible, and where the straights change direction, curves within the limits of permissible curvature may be fitted to the straights. Boxes of railway curves made in transparent celluloid may be obtained for all curvatures required and a suitable one for each case can be rapidly found to fit the topography (see Table XXII., p. 288). Great care must be taken in laying down the straights, to see they pass through the governing points or very near to them. If the line crosses a road at the full height required for headway and construction above the surrounding surface, material will be required for the embankments, which will not be provided if the located line hugs the dotted line too closely. However, as the bridge site is approached the dotted line, which is a surface line, will depart from the located line considerably, running up along the high side of the surface. In order to provide material for the bridge embankments the located line must tend towards the high side of the dotted line. This rule generally applies in the cases of both an over and underbridge. Treating the matter of following the dotted line by a length of located line

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in a common-sense manner, the located line should strike an average having about equal areas between it and the dotted line on either side. Although the located line laid down quickly in this manner will not ensure the absolute balance of material for cuttings and embankments, it will give a distribution sufficiently approximate to prevent the necessity of any large adjustments being made subsequently, when the limits of deviation have to be observed and, still more exacting, the 5-foot limit in levels.

The Levels and Longitudinal Section.—Where the located line has been fully marked on the plan the levelman must take it on the ground and take a line of levels along it. Before doing this the continuous chainage should be marked on the line at every point where it cuts a fence or object easily found on the ground. This can be done sufficiently accurately by scaling on the plan. The object of having these chainages marked on the plan is to enable the levelman in making his notes to assign its chainage to each level taken by him. Many of his levels may require to be taken at changes in the surface the positions of which have to be determined by taping from their nearest points on the plan at which chainages are given. Thus the data for a longitudinal section of sufficient accuracy will be obtained, the levels being checked on every bench-mark which exists within reasonable range of the line. In order to place himself on the line marked in pencil on his plan, it will frequently be necessary for him to scale from the nearest corner of a field or from a gate shown on the plan or some definite object easily recognizable both on the ground and on the plan, and tape the distance on the ground. For this reason it is as well for him to be provided with a scale to be used on the plan. In getting the levels for this longitudinal section the levelman is not expected to take levels at every chain length as on a pegged-out line, but only at occasional points, as where the line cuts the fences and points of decided change in the slope of the ground. All points where roads, rivers, or other railway lines are crossed must be shown accurately on the section.

The data now obtained in the field will be taken into the office and the longitudinal section plotted to the same horizontal scale as the plan, and a vertical scale of 100 feet to an inch. On this longitudinal section the gradients are now fixed. Special care must be given to this work in order to improve the line as much as possible. But if at any point there is doubt about being able to secure so flat a gradient as is attempted on the section, it is better to show the steeper gradient and improve it afterwards than to show the flatter gradient and then find it impossible to establish it on the ground with the gradient limits permitted.

The Government Standing Orders say gradients may be diminished to any extent, gradients flatter than 1 in 100 may be made steeper to the extent of 10 feet in a mile, and gradients steeper than 1 in 100 may

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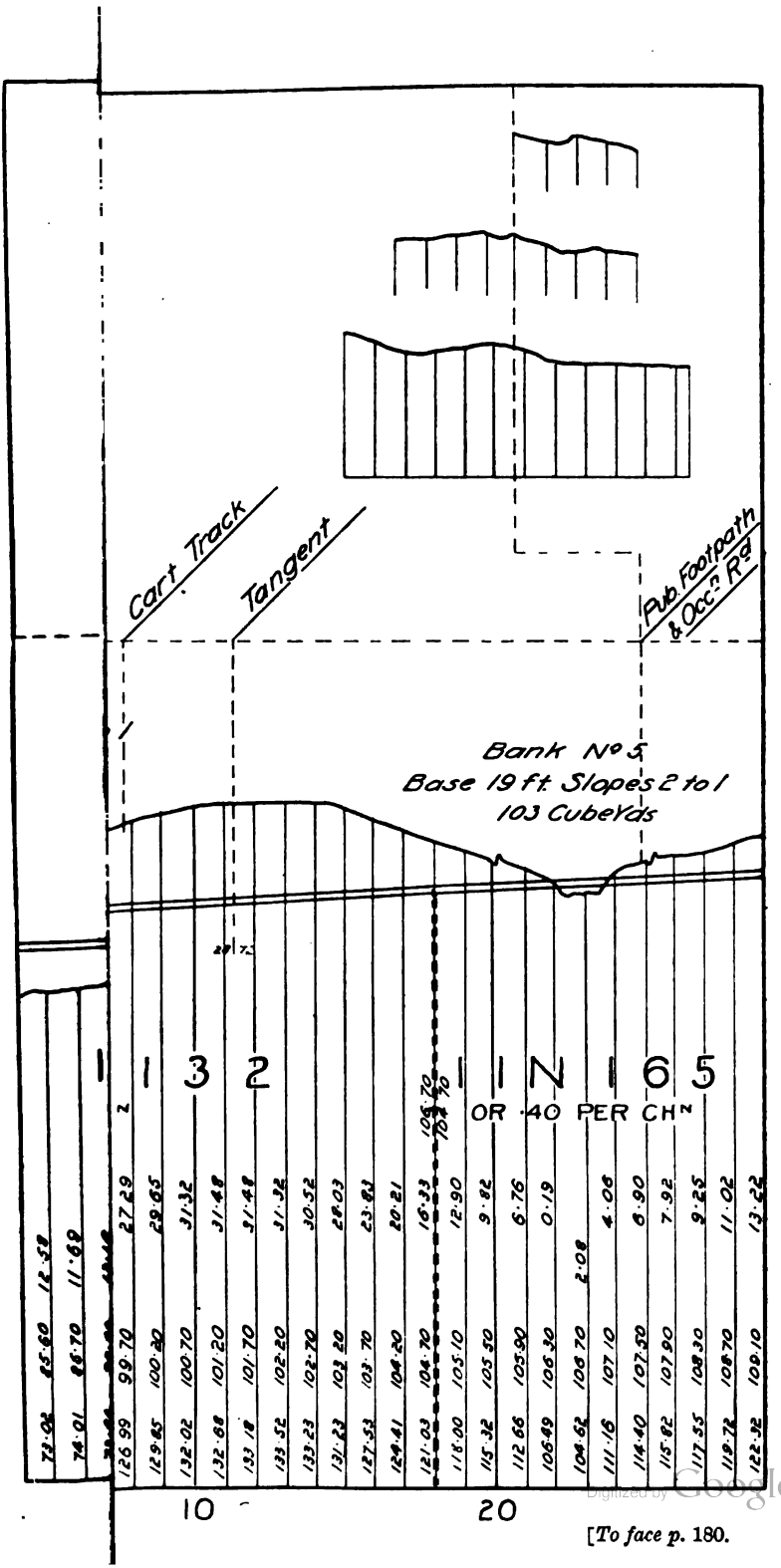

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be made steeper to the extent of 3 feet in a mile or to such further extent as may be authorized by the Board of Trade.

Parliamentary Plans.—These are usually prepared from the 6-inch ordnance plan on which the line has been located by tracing. The Standing Orders require the plan to be to a scale of not less than 4 inches to the mile, consequently the 6-inch plan will serve the purpose well. But every garden and building must be shown to a scale of 400 feet to 1 inch. Accordingly it is customary to show these to the required enlarged scale below the general plan, but above the longitudinal section which is also shown on the same sheet beneath the plan. If much of the land through which the 600-foot wide strip passes includes gardens and buildings it may be convenient to make the whole general plan to a larger scale, say the 25-inch ordnance. A dotted line usually marks the limits of deviation (see Fig. 26, p. 180). Within the limits of deviation each field, garden, building, road, etc., must be marked with a reference number on the plan, and the numbers are also entered in a book together with the owner's name and a description of the property. The chainage must be marked on the plan in miles and furlongs. The longitudinal section shown under the plan on the Parliamentary sheet is also prepared by tracing from the longitudinal section already prepared. This also conforms with requirements which are:— The section must be drawn to the same horizontal scale as the plan and to a vertical scale of not less than 100 feet to an inch and must show the level of the ground, the level of the proposed work, the greatest height of every embankment, the greatest depth of every cutting and a horizontal datum line which shall be referred to some fixed point, near one of the termini. The distances must be marked in miles and furlongs to correspond with those on the plan. The rate of inclination of each gradient must be shown, and the vertical height of the rail level above datum at each change of gradient and at the points of commencement and termination of the railway must be marked on in feet and decimals of feet.

Where the line crosses any public road, river, canal or railway, the difference in the heights of the respective surfaces must be marked on in figures and if a surface is altered the amount it is raised or lowered must be marked on, also the span and headway of each bridge.

There are many other regulations in the Parliamentary Standing Orders which will need to be studied before railway Parliamentary work should be undertaken. Also it will be as well to study the requirements of the Board of Trade, many of which have important bearing upon the surveying and "setting out" work.

The Estimate.—An estimate of the cost of the railway must be provided on a special form which requires the amounts of earth-

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work in cuttings in various materials, the cost per cubic yard and total cost;

The number of cubic yards in embankments and cost;

The number and cost of bridges, also of accommodation works;

The amount and cost of the permanent way, also of land and buildings. Numerous other headings are provided so that the estimate may be as complete as possible.

Final Location

When sanction has been obtained for the Parliamentary Scheme, more accurate work must be done in determining the best position for the line from the point of view of economy in construction.

As the Parliamentary line was laid down with some care on account of the restrictions to its being altered very much afterwards, it may be used as a base line from which to take rough cross-sections.

If the 25-inch plan was used as a Parliamentary plan it can be used for showing the contours, but if the 6-inch plan was used upon the Parliamentary Scheme, a 25-inch one must be prepared and the Parliamentary line plotted on it.

These plans should be cut in strips from 12 to 20 inches wide, mounted on linen and folded in book form. The 100-foot contours from the ordnance map must be plotted on this plan.

Field Work.—The plan must be now taken into the field and the centre line poled out, a few lengths at a time, small pegs being used to mark their positions. There is no need for a theodolite at this stage, as the ranging poles can be lined in by sighting from one end. The position of the line will be determined by scaling on the plan from recognizable points on the ground and taping off the distances. No continuous chainage need be made along this line, but points where cross-sections will be required should be marked with pegs. The positions of these must be taped on the ground to the nearest object shown on the plan, so as to mark them with some degree of accuracy.

It is advisable to number them, as their continuous chainage is not determined. The levelman now takes levels at these points, entering the number of the peg and its level in the level book. When a number of these levels have been checked on the nearest bench-mark and their reduced levels worked out, cross-sections may be taken with a hand-level, staff, and tape. Only contour levels are required on these cross-sections, the reduced level of the centre point being known. As the deviation lines are at 300 feet on each side of this centre point no contours need be fixed beyond that limit, and generally it is sufficient to fix two or three on each side of the centre and estimate the positions of any other by clinometer or by eye.

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Plotting the Topography in the Field.—The method of fixing contours has been described on pp. 51 and 52. In this case it is not necessary to plot them on square-ruled paper, as they can be plotted directly on the plan. Fine pencil lines should be drawn on the plan parallel to the deviation line and at 100 and 200 feet respectively from the centre line on each side of it. With the help of these lines the positions of contour points taped from the centre can be estimated by eye. For example, if contour 95 occurred at 34 feet it can be dotted in at one-third of the distance between the centre and the 100-foot line. Similarly 58 feet is as much inside the two-thirds mark as it is beyond the half-way distance. The distance on a 25-inch plan between the centre and the 100-foot line is less than half an inch, consequently little skill is required in placing these contour points in their approximate positions by eye. It is not necessary to mark at these dots on the plan the contour levels except on the first section taken, as at each subsequent section each contour line will be sketched on the plan from its position on the previous section whilst the topographer is looking at the ground, thus completing the sketching in of all contour lines up to the section just taken. As sections in ordinary country will only need to be taken on an average every 300 or 400 feet, this work can be done very rapidly, 3 or 4 miles being easily completed in a day.

Final Location of the Line.—When 20 or 30 miles of line have been prepared in this way on the plan, the final location of the line can be determined in the office by trial (see p. 66) and the gradient lines fixed upon the longitudinal section plotted from the plan as described on p. 68. In performing this location care must be taken to keep the final line very close to the governing points on account of the limits imposed. The balancing of cuttings and embankments by some method of distribution of material must be decided at this stage. When this work has proved satisfactory the next operation is to peg the line out on the ground.

Pegging-out the Centre Line.—This operation is performed by taking points on the located line on the plan, the positions of which can be easily fixed on the ground by measurement from near prominent objects such as the corners of buildings or other objects or fence-lines and marking out a number of these points. As many points on each line as possible should now be fixed in this manner on the ground and ranging poles set up at them. For each length of straight the complete set of points should be dealt with at once. When sighting along the line of these poles it will be found that they are not all in line on account of the scale of the map and the method which had to be resorted to to fix them. An average line through them must be adopted, some being moved a little to the left and others to the right. If the line is

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a long one, and rising ground near the centre prevents their being all seen from one end, they must be observed from the highest point and it will be necessary to set up a theodolite at this point for the purpose. No poles must be changed until the two parts of the line are sighted upon by the instrument, which must be in good adjustment on account of the risk of attempting to produce a straight line by transiting the telescope. If it should be found that the position selected for the instrument does not give a good average line for all the points, it must be moved to the right or left. But before doing so, an observation should be made to determine by how much the instrument should be moved in order to avoid making too many trial settings-up.

When two lines of straight have thus been ranged out and the ranging poles adjusted into a good line, their point of intersection must be carefully found and a large square peg driven at it. The instrument will now be set up on this and sights taken along both lines to check the accuracy of the intersection point. Chaining or taping with a steel tape can now be started from the beginning and large square pegs driven flush with the ground every 66 feet at points accurately lined in by the instrument, the positions of tangent points for the curves must be calculated, and the curves ran in on the ground as described in Chapter VII. The radius of the curve on the plan was determined by scaling, but it may be found this does not exactly fit the topography on the ground as it appeared to on the plan; consequently it may be necessary to change it. This should be done at once so that the continuous chainage may be correct. Until the levels and cross-sections are taken it will be advisable to mark every 66-foot station on the right side of the peg with a label of wood, on which is marked the chainage, driven in a few inches into the ground. In this way the whole line will be set out on the ground. The pegs used for station points should be 2 inches square and about 18 inches long. At points of intersection, which should always be preserved tangent points and intermediate points in long lengths of straight, these pegs should be centred by having a small flat-headed copper nail driven in them. Tangent points should be marked by guard pegs one on each side of the centre line, driven a few inches from the peg with their points sloping towards it and should be left projecting 2 or 3 inches above the ground. All tangent points should be marked by reference pegs, see p. 96.

Completing the Survey.—The centre line pegged-out on the ground now serves the purpose of a base line or traverse line from which measurements can be taken to any new artificial topography not shown on the 25-inch plan and thus enable the latter to be brought up to date. For these and the following data a field book is now prepared. The chainages of all points cut by the line such as fences, roads, etc., are also recorded, and complete surveys made of the sites and approaches to

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all bridges and of the areas within which road or river diversions are to be made or at points where any special structural work is to be carried out.

The Longitudinal Section.—Levels are taken on every peg driven on the line staked out and at intermediate points where the surface changes. Bench-marks are also fixed about every half-mile on tree trunks or other objects beyond the possible limits of earthwork so that they will be preserved during construction. The section is now plotted to the $\frac{1}{2500}$ or 25·344 inches to the mile scale for horizontal lengths and 30 feet to an inch for vertical heights. Upon this section a final gradient line must be laid down, and when the cross-sections are taken the actual amount of material in cuttings and embankments can be studied and the manner in which the material should be distributed. (For the most systematic way of dealing with this see pp. 75 to 82.) It will probably be found that the line conforms very nearly with that required, but here and there a little revision will be necessary. Great economy results from even small revisions, so no attempt to stint the time spent on revisions is advisable, as it is very unlikely that the cost of the revision will approach anywhere near the saving in the cost of construction.

Any parts which have been revised on the plan will need re-setting out on the ground, and all the subsequent part of the line may need re-pegging or re-numbering to make the chainage correct.

Cross-Sections.—These need not be taken at every chain peg unless the ground is very irregular, but even then more satisfactory results are obtained by taking them only at changes in the surface of the ground or at points which will give a fair average change of the actual surface. The level on the centre line of each cross-section must be taken if its reduced level is not already obtainable from the longitudinal section.

The cross-sections should be plotted to the same vertical scale as the horizontal scale to facilitate in calculating their areas.

The side slopes and formation lines in cuttings and embankments must also be plotted on these sections and sometimes the positions of the fence lines are shown. Side ditches must also be plotted on.

Any revision of the line will probably be within the limits of the surface cross-sections, the formation line and side slopes being corrected to suit the new line.

The Working Drawings

Plan and Longitudinal Section.—When the cross-sections have been corrected the new positions of the fences should be marked on

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The half-widths or distances from the centre line to these on each side will be scaled off on the 25-inch plan and marked for every cross-section taken. A line is then dotted in, connecting these with a smooth curve, thus showing the railway boundaries on the plan. The chainage is marked in miles and every 5 or 10 chains, but a tick is made on the side of the line for every chain point. The chainages of tangent points are also usually marked on and the number of chains radius for each curve shown on the plan. Where bridges occur a reference is generally given to the number of the drawing giving details. The chainages of the bridges are generally mentioned, each one being described by its chainage, *e.g.* the bridge at 3 miles 49 chains.

Any other special works, for example diversions, accommodation, works, etc., are generally described in a similar manner, and if a special drawing is provided, its number referred to. Fig. 62 gives a short length of working plan used in the construction of a railway in the south of England.

The longitudinal section corresponding to the above-mentioned plan is shown on Fig. 63. The chainage on these sections is marked the same as on the plan, the horizontal scale being the same for both. The vertical scale is 30 feet to an inch. This working section is prepared from that already described by tracing. All levels are referred to the ordnance datum, but the datum along the base of the plan is probably only 40 or 50 feet below the lowest point to be found on the surface of the longitudinal section. Vertical lines are drawn at every chain point and on these is marked the surface level and immediately above it the depth of embankment, if any, is given. A corresponding space above this is used for giving the depths of cutting. The chainages of all bridges, occupation works, etc., are marked on the section on vertical lines above the surface line. Where the levels of roads crossing the railway are changed or new road-approaches to be made to bridges or occupation bridges, a section of the road showing the amount of embankment or cutting at each chain point on its centre line is frequently given on the longitudinal section above the general section.

Gradients are marked on in the usual manner and also the amount of change in feet per chain. For example, a gradient of 1 in 132 would also be given as ·5 feet per chain. This is very convenient to a foreman in charge of an excavation gang, as in such a case he can bone-in his formation levels by raising his sights six inches at each chain point, without any calculations. Points of change in gradient are marked by a double dotted line. The chainages of tangent points are also marked on the working section. The amount of material or volume of each cutting or embankment is marked above it on the section together with its number, the width of the base at formation, and the slope of the sides.

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CHAPTER XI

SETTING OUT WORK FOR CONSTRUCTION

THE first demand upon the resident engineer or his assistant, when starting construction, is made for slope stakes, commonly called "batter pegs" in this country, unless these pegs have been fixed when cross-sections were made. It is customary to provide the foreman excavator or the "walking ganger" with a copy of the working longitudinal section on a roll of tracing cloth or in book form. This section provides him with the depth of cutting or embankment at each chain peg and also the rate of gradient per chain, so that he can set up sights for boning-in his formation levels. He checks his depths by measurement at each station peg, comparing them with those given on the longitudinal section and can detect any appreciable error in the level of a chain peg by the depth at it not agreeing with that given on the section. When his gang has started to take out the centre width of a cutting, especially if the depth is not large, he will require immediately batter pegs at each chain-peg section. These pegs are at points on the surface where the side slopes outcrop. When the formation is excavated to its full width, the slopes will be excavated true to slope by means of batter rules, but the batter pegs on the surface are essential to check the accuracy of the batter rules. They are also frequently used to enable some members of the gang to be engaged dressing the slopes down from the top, whilst others are dressing them up from the bottom.

Batter Pegs or Slope Stakes.—Fig. 64 shows a cross-section of a cutting, AED being the natural surface of the ground, in this case considered level. E is at the centre peg or chain point. F vertically below E, is the centre of the cutting at formation level, BF and CF being the half-widths a.

BA and CD are the side slopes, their rate of slope being S. That is

$$\frac{GB}{GA} = S = \frac{CH}{DH}$$

EF = depth d. EA = b_l the left half-breadth and ED = b_r the right half-breadth.

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Since in this case the surface is level EA = ED.

The rate of slope S is commonly expressed as $1\frac{1}{2}$ to 1, 2 to 1, $\frac{3}{4}$ to 1, or some such ratio, the first figure of each ratio representing the horizontal distance and the second the vertical height. A list of these ratios for different materials is given on p. 195.

In Fig. 64 since the ground is level AG = EF = d

$$\therefore \text{GB} = \text{S} \cdot \text{AG} = Sd$$
$$\therefore b_l = \text{EA} = \text{GB} + \text{BF} = Sd + a$$

Similarly in this case $b_r = Sd + a$

Therefore if it is known that the surface AED is level the batter pegs at A and D can be easily fixed by taping from E a distance equal to a, and adding to this a length equal to the depth × slope.

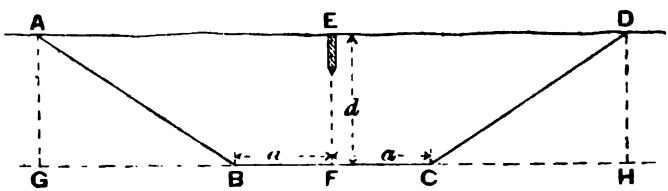

FIG. 64.

For example, if the depth of cutting at E is 8 feet and the side slopes at $1\frac{1}{2}$ to 1, and if a the half-width is 15 feet, AE = 15 + 8 × $1\frac{1}{2}$ = 27 feet and the points A and D are each at 27 feet from E.

However, it is usually difficult to say at a glance that the surface is level, consequently it is customary to take a reading with a level on a staff placed on E, and if the ground looks level to calculate the half-breadth for a level surface and test the reading on the staff at the point so found. Then, if the reading at A is found to be so very nearly the same as that at E that a movement of two or three inches out or in will adjust it, no further work will need to be done in fixing the batter pegs.

Under similar conditions the pegs marking the toe of the slope of an embankment may be fixed by a similar operation, the distance out from the centre peg being: the half-width at formation together with the depth multiplied by the slope ratio, the condition being the same as in Fig. 64 inverted.

Batter Pegs on Side-slope Ground.—But when the surface slopes from one side to the other, the operation of finding the position of the batter pegs is more difficult.

In Fig. 65 the surface slopes from D down towards A, consequently the side slope CD must extend a longer distance to reach the surface at

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D than in the case of the level section ; whilst on the down side the slope from B, instead of extending to a point A on the same level as E, is met by the surface much sooner and consequently AE is a shorter "half-breadth" for the same depth d, than in the level section. In this case the half-breadth ED cannot be given as $a + Sd$, because $DH > d$.

The operation of fixing these points is as follows :—Set up the level at a point from which it will be possible to get a reading both on the high side, as RD, as well as the low side QA, and now read the staff on the peg at E. Let this reading be r, then the height of the instrument or collimation level above the formation is equal to $d + r$ and the horizontal distance $ED = a + S(d + r - y_2)$. Similarly $EA = a + S(d + r - y_1)$. Now, in attempting to fix the point D for the batter

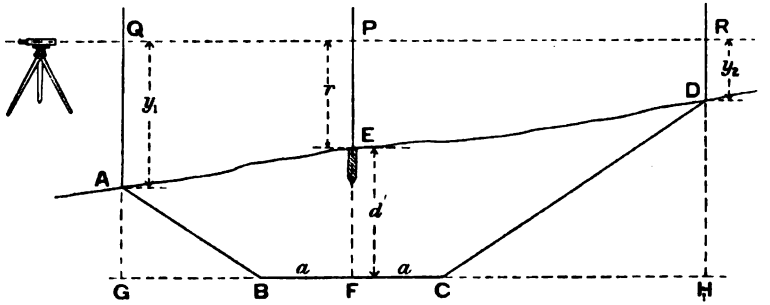

FIG. 65.

peg on the right a length greater than that for a level section is tried. For example, if ED, for a level section, was equal to 27 feet and by eye the surface seemed in that distance to rise about 3 feet, if the side slopes are at $1\frac{1}{2}$ to 1, a distance equal to $27 + 3 \times 1\frac{1}{2} = 31{\cdot}5$ feet might be taken as a first trial breadth, though the very fact that the ground is rising suggests that in the last $4\frac{1}{2}$ feet the surface level will have changed sufficiently to require a further extension of the half-breadth. If the ground continues to rise at the same rate, say 3 feet in 27 or 1 in 9, in $4\frac{1}{2}$ feet the ground will rise ·5 feet, which will demand an additional length of $\cdot 5 \times 1\frac{1}{2} = \cdot 75$ feet or 9 inches. And again, in the distance 9 inches, the ground rising at the same rate, a further change of 1 inch in level will occur which will demand a further extension of $1\frac{1}{2}$ inches in length. Although the half-breadth should still be extended if mathematical accuracy is to be obtained, in fact this system of additional lengths is an infinite series, no practical advantage can be gained in so doing, and when it is seen to be within an inch or two of its correct position the peg will be driven 1 or 2 inches farther out without further testing the surface level. In the

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above case an expert would probably try 1 foot more than 31′ 6″ and thus hit off the correct position to an inch on the first trial, that is provided a level was taken at 27 feet giving 3 feet difference.

Batter Pegs on Uniformly Sloping Ground.—Although it is not recommended to make a practice of determining the average slope of the ground, on account of its tending to delay operations, still on uniformly sloping ground the true position of the batter peg can be accurately calculated after taking level readings on the centre peg and one other point, either at the first approximation to the batter peg point or at the distance for half-breadth of a level section. The distance tried divided by the difference in level gives the surface slope. Then the distance tried divided by the surface slope and multiplied by the side slope gives the additional distance out to be tried.

Thus if l = the distance tried and $f = \frac{\text{side slope}}{\text{surface slope}}$, the total length is the sum of the following geometrical series :—

$$\text{Total length} = S = l + lf + lf^2 + \ldots + lf^{n-1}$$

The sum of this series is $S = \frac{l(1 - f^n)}{1 - f}$

But as f is a fraction, generally small, and n is ∞, the sum becomes $S = \frac{l}{1 - f}$

This formula can accordingly be used for determining the distance to the batter peg point.

For taking the example just given—

$$l = 27', f = \frac{1\frac{1}{2}}{1} \div \frac{9}{1} = \frac{1}{6}$$

$$\therefore S = \frac{27}{1 - \frac{1}{6}} = \frac{162}{5} = 32{\cdot}4'$$

which is the correct distance from the centre peg to the batter peg.

In above example the trial method gave for the first four terms of the series $27 + 4{\cdot}5 + {\cdot}75 + {\cdot}125 = 32{\cdot}375$, which gives a total only $\frac{1}{3}$ of an inch less than the correct total 32·4.

It must also be noted that the first three terms give a total of 32·25, only 2 inches less than the correct length 32·4.

However, unless the side slope of the ground was very uniform this method, *i.e.* using the formula, will prove of little advantage, the common and most convenient method being that of finding the first 2 or 3 terms of the series or what may be called the trial and error method.

General Problem of Locating Batter Pegs.—The following

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example will illustrate most clearly the general method of locating batter pegs:—

The centre depth is 18·6 and the half-width is 10 feet (see Fig. 66). Therefore for a level section the half surface breadth will be $10 + 18{\cdot}6 \times 1\frac{1}{2} = 10 + 27{\cdot}9 = 37{\cdot}9$ feet. Having calculated this amount, a glance at the surface may suggest a change in level of 2·5 feet at this distance, thus requiring the distance to be increased at least $2{\cdot}5 \times 1\frac{1}{2} = 3{\cdot}75$ or say a total of 42 feet. At this point a reading of 6·3 is obtained or a difference from the centre $= 10{\cdot}2 - 6{\cdot}3 = 3{\cdot}9$. Now 3·9 will require $3{\cdot}9 \times 1\frac{1}{2} = 5{\cdot}8$ feet in addition to 37·9 or 43·7. Thus even at 43·7 it may be expected the surface will have risen more than at 42, say 1 foot, thus making a length of 45·2 still too small. It is generally advisable to try a couple of feet more, say 47. At this distance the reading is 3·5, giving a difference of 6·7 and requiring the distance from

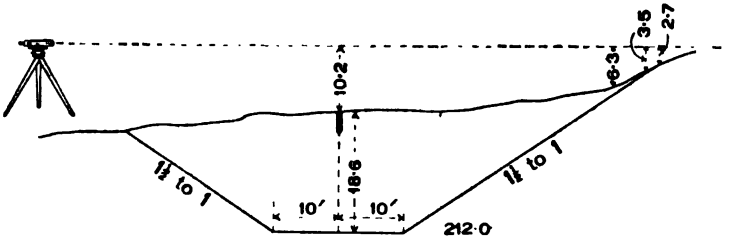

Fig. 66.

the centre peg to be $37{\cdot}9 + 6{\cdot}7 \times 1\frac{1}{2} = 47{\cdot}9$ or about a foot more than that taken. It is now very near the point, though the ground here is changing level rapidly. The change in level cannot be more than 6 inches in the next foot, so 49 feet ought to determine the true position. At this distance the reading is 2·7, giving a difference of 7·5 or requiring a total distance of $37{\cdot}9 + 7{\cdot}5 \times 1\frac{1}{2} = 37{\cdot}9 + 11{\cdot}25 = 49{\cdot}15$ or 49′ 2″; without taking further level the peg would be fixed at 49′ 3″, giving 1 inch extra for any additional rise in the last three inches.

This may seem a slow operation, but to one accustomed to perform such work, it will be done very rapidly; also a section rarely occurs where 3 levels will require to be tried as in this case, the ratio of slope on the same side at a number of chain points tending to be the same and thus enabling the true position for the peg to be found frequently within 6 inches at one trial.

On the low side in the above example it will be found that the trial breadths will tend to overrun the point and a shorter one may require to be tried to approach the true position.

In the case of embankments the operation on the low side is very

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similar to that on the high side in cuttings, the difference in level between the centre and the trial point suggesting how much farther than the level section half-breadth the point must be carried out.

The case of half cutting and half embankment should be carefully observed. In Fig. 67 the centre depth shows embankment only and the left batter peg will be set out in the ordinary way. But on the right it will be found, if not observed before, that the difference in the level of the centre peg and that of the trial level section point is greater than the centre depth of the embankment, and consequently the point on the surface is above formation level instead of below it. It is, then, advisable to find the point on the surface where the difference in level is the same as the centre depth. This point is the intersection of the surface with the formation level and should be marked with a peg, as it is useful in marking where embankment changes to cutting. Then

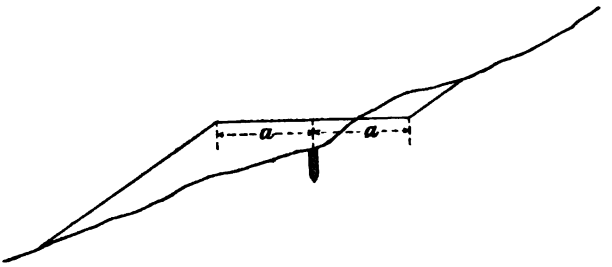

FIG. 67.

in fixing the batter peg a trial level at the half-width a, will suggest by its height above formation level how much farther out the probable point should be found, the operation being then the same as for ordinary cuttings. It is strongly recommended to draw a sketch of the cross-section, if any doubt is felt about the locating of this point.

The operation of finding batter points by the method here set out may not seem to some to be very scientific, but the results obtained are those which are required and the speed in obtaining them is surprisingly greater than the description of the process would suggest.

Other methods for which formulæ are obtainable are dependent upon the surface slope being determined. But as this slope may change several times in one cross-section the operation becomes very cumbersome and liable to error.

Slope Stakes for Cuttings in two kinds of Material.—In setting out slope pegs care must be taken to find out if there is any likelihood of the nature of the material changing in the full depth of the cutting. Fig. 68 shows a depth of clay at the surface overlying rock. Partly a guard against such a condition occurring without being

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anticipated and thus not prepared for, it is usual to excavate a central gullet not wider (generally much narrower) than the formation width down to 6 inches or 1 foot above formation [but chiefly to enable 1 or 2 wagon roads to be made for the purpose of facilitating the removal of the excavated material]. This reveals the nature of the material down to formation.

If rock is struck, as in Fig. 68, it will be necessary to clear the top surface of it for a width a little greater than the formation width and greater still as the depth of rock increases. On this cleared surface levels will need to be taken to determine the true depth of the rock to formation and then batter pegs (or iron dogs) driven into the rock at the batter points for the rock side slope ($\frac{1}{2}$ to 1 in Fig. 68). The clay slopes are supposed to start 2 feet back from these points and run at

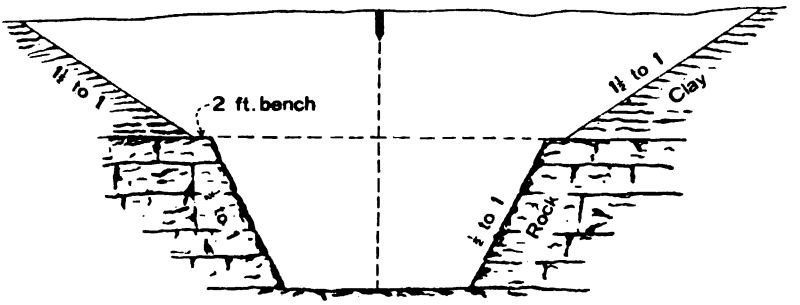

FIG. 68.

their natural slopes up to the top surface. This 2-foot bench or berm is provided to prevent the clay slope tending to slide over the edge of the rock. [It is usual to provide a wider bench than 2 feet if the clay slopes are not sown with grass seed.] The calculation for the position of the batter peg on the surface of the clay is made by determining thedifference in level between the surface of the bench 2 feet back from the rock batter peg and that at the trial point on the surface of the clay and multiplying by the rate of slope in the usual way.

If batter pegs had been originally put in for clay alone, their positions in cuttings especially deep in rock will be much farther out from the centre point than the correct pegs. Care should be taken to remove the original batter pegs, in case any mistake should be made in trimming down the clay slopes from the top.

Squaring off for Cross-sections.—The operation of squaring off at each chain point is usually performed by means of a cross-head or an optical square, ranging poles being put in on the line of the cross-section. In squaring off on a curve the tangent offset for a chain length should be calculated and a ranging pole fixed at that distance from the

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next chain peg on the outside of the curve. The cross-head is then made to sight on this pole and the square line marked out at right angles to that direction (see p. 209).

Taking and Recording Cross-sections Notes.—If accurate cross-sections have not already been made of the railway, they can be quickly taken at the same time as the batter pegs are being fixed. These may be recorded as shown in the table below, chainages being written from the bottom of the page upwards.

Plenty of space should be left for the cross-section part in the table, as in the event of the ground being very irregular a number of surface points will need to be taken. Fig. 69 shows a cross-section at chain 17

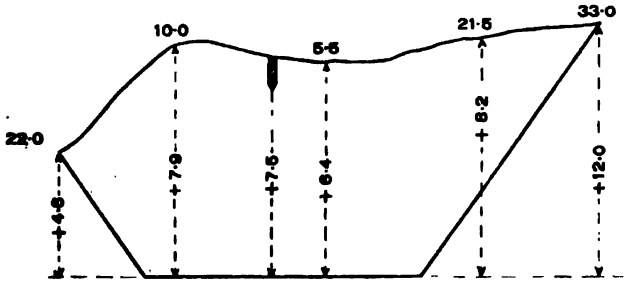

FIG. 69.

in the table. By comparing dimensions shown on figure it will be seen that the distance out from the centre on each side and the level at that distance are represented in the table in a fractional form, the distance being in the numerator and the height from formation in the denominator. This is marked by a + sign when the point is above formation as in cuttings and by a − sign when below formation in embankments.

TABLE XIII.

FORM OF CROSS-SECTION NOTES.

Chain point.	Surface level.	Formation level.	Cross-section. Left.	Centre	Right.
21	208·9	210·2	27·3 / − 8·2	− 6·8	19·6 / − 3·1
20	205·8	209·6	21·9 / − 4·6	− 3·8	17·1 / + 1·4
19	209·6	209·0	16·2 / − ·8	+ ·6	23·4 / + 5·6
18	212·1	208·4	20·7 / + 3·8 ; 12·1 / + 4·0	+ 3·7	19·5 / + 6·5 ; 28·2 / + 8·8
17	215·3	207·8	22·0 / + 4·6 ; 10·0 / + 7·9	+ 7·5	5·5 / + 6·4 ; 21·5 / + 8·2 ; 33·0 / + 12·0
16	217·9	207·2	23·3 / + 5·5 ; 6·0 / + 11·2	+ 10·7	18·0 / + 7·9 ; 31·5 / 11·0

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These notes should be entered on the left-hand page of the field note-book, the right-hand page being used for calculations, trial levels, etc.

SAFE RATES OF SLOPES IN CUTTINGS IN DIFFERENT MATERIALS

Material.	Rate of slope.
Compact earth and well-drained clay	$1\frac{1}{2}$ to 1.
Dry London clay	2 to 1.
Wet clay	From 2 to 1 to 4 to 1.
Dry sand	From $1\frac{1}{4}$ to 1 to $1\frac{1}{2}$ to 1.
Gravel	From 1 to 1 to $1\frac{1}{2}$ to 1.
Solid rock (with strata not dipping into the cutting)	$\frac{1}{4}$ to 1.
Solid rock (with strata dipping)	From $\frac{1}{2}$ to 1 to $\frac{3}{4}$ to 1.
Soft and loose rock	From $\frac{1}{2}$ to 1 to $\frac{3}{4}$ to 1.
Chalk	From $\frac{1}{2}$ to 1 to 1 to 1.

In embankments the slopes in most cases are the same as above. Rock, however, is usually tipped at $\frac{3}{4}$ to 1 to 1 to 1.

Fixing Fence Pegs.—These are usually driven at a fixed distance from the top of the slope in cuttings and the bottom of the slope in embankments. This fixed distance varies for different railways and under different conditions from about 6 to 12 feet. It is customary to have a wider strip, say 10 or 12 feet, inside the fence on the high side for the purpose of providing a side ditch to carry off the surface water. This is imperative in the case of embankments.

Fence pegs should be set out by measuring the fixed distance from the batter pegs. As in undulating country batter pegs follow a very winding course, the fence pegs, as set out, will not form a very good line. Consequently, ranging poles should be set up at the fence pegs as set out and a number of them at a time eyed-in to form a line of easier curves. This is done by moving some out a little and others in so as to enclose about the same area as before.

Borrow Pits.—Borrow pits take various forms. The material required is sometimes obtained by widening a cutting, in which case the additional batter pegs will require to be fixed in the ordinary way for the increased width. It will be necessary to take a cross-section of the surface so that the amount of material obtained from the extra width may be known. Again, the material to form the embankment may be obtained from side cuttings or trenches as shown in Fig. 70.

These must be separated from the toe of the embankment by a sufficient space or berm not less than 6 feet, being more for high embankments. They should be regular in form and drained if possible to prevent water standing in them. Batter pegs marking the tops of the

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side slopes of them should be set out at the same time as those for the embankment, also levels for the cross-sections should be taken over the surface if irregular and the depth of excavation from centre pegs fixed for each trench to ensure a uniform fall from one chain to the next to enable them to be properly drained. Care must be taken in fixing these centre pegs to ensure that the berm is kept sufficiently wide at every point.

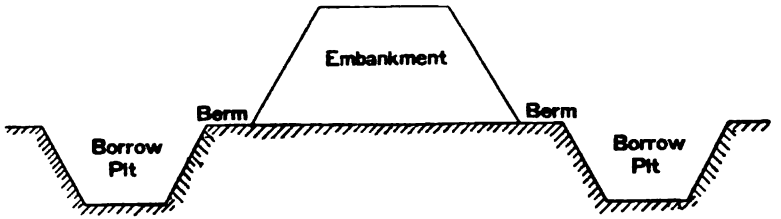

FIG. 70.

SETTING OUT BRIDGE FOUNDATIONS

Fixing the Centre Lines.—Before attempting to mark out the ground for the excavation of a bridge foundation, both the centre line of the railway and the centre line of the road or river must be accurately fixed. No pegs existing on the ground as centre pegs must be accepted as correct without very carefully checking the line. The centre line of the road must also be carefully fixed. This may involve a considerable amount of work, as even the line of irregular fences along a road must be averaged and the road tested for its correct width on both sides of the proposed bridge before any attempt to fix the correct centre line can be made. Nothing throws more discredit upon this "setting out" work than that the face line of the bridge, instead of being true to the direction of the road, should run oblique to it, thus making the finished line of road after construction follow the proverbial "dog's hind leg" alignment. After carefully determining the true centre line of the road at the centre of the bridge pegs must be driven to render it as permanent as possible. If these require to be in the surface of the road, iron dogs or large nails will be needed in order to make it possible to drive them. When driven the true line should be marked on the top of them with a file. At least two should be driven in each direction from the centre of the bridge (see Fig. 71), one in each case being as near as possible to the bridge for convenience in "setting out" work, and the others at a greater distance to ensure their being undisturbed. All these pegs should be fixed by lining them in with a theodolite from one end. A centre peg must also be fixed; this can be done by the theodolite lining it in on this centre line of road,

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provided the centre line of railway has already been fixed. (This line should also be fixed by two pegs in each direction as a check against

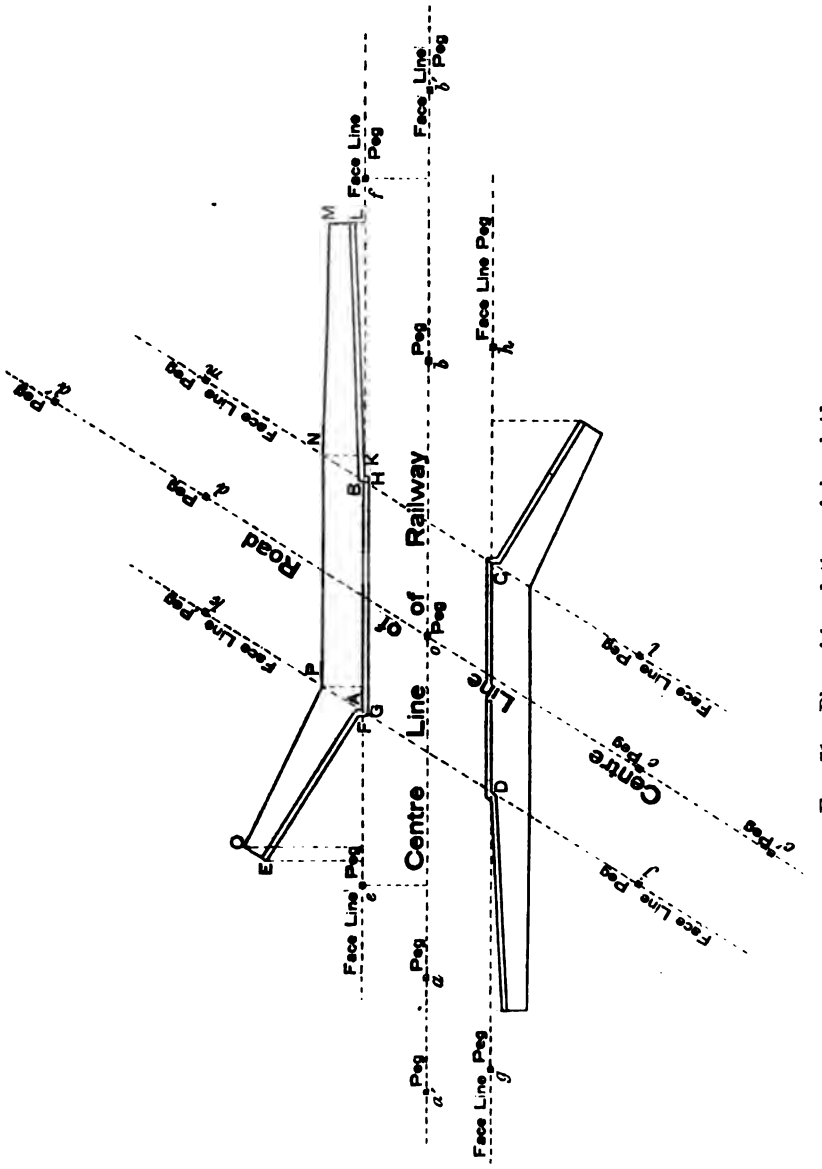

FIG. 71.—Plan of foundations of skew bridge.

their being disturbed.) A string line must be strained over the nearer pegs *a*, *b* (Fig. 71), and the theodolite sighting along the centre line

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of road intersects with the vertical cross hair the string line vertically above the exact position of peg *o*, which is then driven and the string line again strained to enable the exact centre point to be marked on the peg.

The theodolite is now moved to the centre peg *o* and the angle of skew of the bridge measured. If the bridge has already been designed and working drawings prepared, this angle should be compared with that given on the drawing. If these angles agree very nearly, the bridge must be constructed to the angle given upon the drawing. This may render it necessary to change the centre point of the bridge by some small fraction and adjust the centre line of the road accordingly. But if the angles do not agree sufficiently nearly, it may be necessary to have the drawing altered to suit the correct angle of skew.

Fixing the Face Lines.—Having set out the centre lines, the face lines of the finished work must next be fixed. A string line is now stretched and fixed on the centre line of railway and a wooden square, say 6′ × 4′, or even a smaller one 4′ × 3′, laid very near to it with one edge parallel to it at one end of the string line. Then a steel tape is held with its edge parallel to the other edge of the builder's set-square, with its zero on the centre line and the point of half-width of the road in feet and inches held at the other end. At this latter point a peg is now driven as *e* in Fig. 71. When this is properly driven flush with the ground, the half-measurement of the road is taken again and a nail driven in at the exact distance. The same operation is carried out for peg *f*. Both these pegs must be fixed clear of the excavation and if possible at a sufficient distance from it to prevent their being covered by excavated material.

A corresponding face line is now set out for the other abutment with pegs as at *g* and *h* in figure. Lines must now be strung over both these face lines, and the full width of the road tested by the steel tape with the help of the square, to check the accuracy of the work. If this is not exact at every point some small error must have crept in which must be carefully corrected, as neat work must be built to exact dimensions. If the surface of the ground is very uneven at a bridge site, it may be necessary to set up the theodolite on each face line for the purpose of fixing other points on the line.

The face lines parallel to the line of railway having now been fixed, those parallel to the line of road can be fixed in a similar manner. These lines are generally all that is required for the mason or bricklayer to work to, though they do not fix the wings in any way. As the wings generally have battering faces, face lines are dangerous unless accompanied by the levels at which those lines are to be worked to.

Setting Out the Lines for Excavation.—The bridge can now be set out for excavation. The same degree of precision is not required

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in fixing the points marking the lines of excavation, but an error of more than one or two inches should not exist. Pencil lines should be ruled on the drawing giving the limits of excavation required beyond the neat work. If it is known that a good foundation cannot be obtained without going down to a considerable depth, these lines must be clear of the neat work by the width of the projecting concrete (probably 6 inches or 1 foot) and the thickness of timbering necessary to keep the sides from falling in. On Fig. 71 the line of the edge of the concrete is shown as EFGHKL projecting in front of the wings and abutment but flush with them on the back. If the depth is not sufficient to demand timbering, this line of the edge of the concrete should be set out for excavation and the back neat to the unfaced brickwork or masonry.

On the drawing in the office offsets square to the face line *ef* are drawn in pencil to each of the points E,F,G,H,K,L,M,N,P,O, and the lengths of the offsets carefully scaled and marked on, also the distances from some known point (as *e* or the centre of the face of the abutment) to the feet of the offsets on the face line, must be measured and marked on the plan in pencil. The plan is now taken to the site and a string line fixed there from *e* to *f*. The distances to the different offset points must now be taped off and marked with arrows. Then the square is laid on the ground with the right angle at the arrow and one side parallel to the string line and the offset set off with the tape parallel to the other side of the square, to the distance written on the plan. In this way each of the points O,E,F,G, etc., are located and pegs driven at them to within one or two inches from the ground. When all these points are pegged out a string line should be fixed to one and continued round each of them in turn until it marks the complete outline of the whole excavation. It is as well to check one or two lengths on this outline, especially in the wings, whose accuracy often depends upon the accuracy of the excavation. This checking also ensures that no point has been omitted. The ground should now be nicked out along the line of the string with a grafting tool or pickaxe, and the narrow piece of turf removed in this way should not be allowed to get back into the nick.

Giving Levels for Excavation.—In the ground clear of the excavation level pegs should be driven at the end of each wing and one at the centre of the abutment. These should have marked on them, or on small pegs beside them, the depths to which the excavation is to be carried.

Levels should be taken at a number of points on the surface of the area to be excavated and marked on the drawing to enable the quantity of excavation to be measured when completed.

The operation of setting out bridges on the square is a simpler form of that given here, so will not be dealt with.

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To Set Out a Bridge Foundation on a Curve.—In this case the centre of the bridge must not be taken on the curve at *b* (Fig. 72), as in that case the curve of the ends of the bridge would be too near one abutment.

The central offset *be* to the chord *aec* must be calculated, and the middle point of it *d* set out on the ground as the centre of the bridge. The line A*d*B parallel to the tangent to the curve at *b* is now the centre line of the bridge. This ensures that E*b* = C*a* = D*c*, or the curve is distant from each abutment at its tightest point by the same amount.

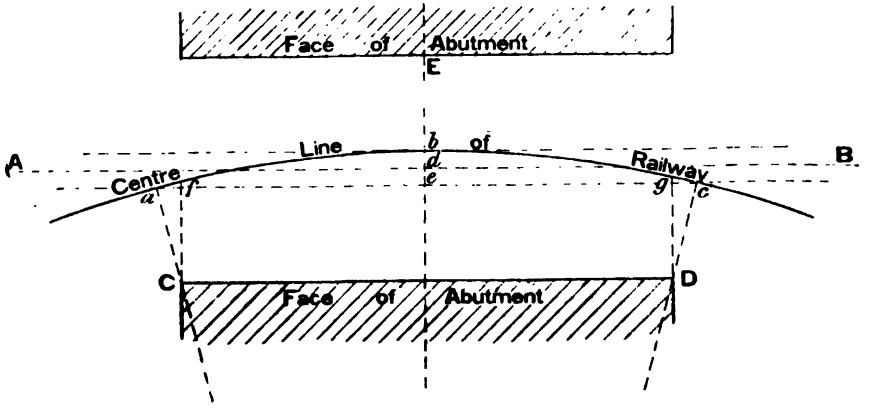

FIG. 72.

Observe that *a*C < *f*C, consequently the length of the face of the abutment is not quite sufficient for calculating the length of the versine *be*.

But the error due to using the length of the abutment for this purpose is very small when the radius of curvature is large.

For a curve of 15 chains radius and length of abutment CD 40 feet, and span between abutments 30 feet, the length of the versine is less than 4 inches.

The offset $be = R - \sqrt{(R^2 - (ae))^2}$ where $\frac{ae}{\frac{1}{2}CD} = \frac{R}{R - \frac{1}{2}(\text{span})}$

R being the radius of the curve in feet. $\frac{1}{2}(be)$ gives the distance from the curve to the centre of the bridge.

CULVERTS

The General Problem.—It is not usual to make a special survey for culverts, type drawings being generally supplied, the actual survey being made at the time of "setting out." The centre line of the

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railway is used as a base line from which to survey the course of the stream. On the plan prepared of this survey the positions of adjacent batter pegs should be given. Levels must be taken in the bed of the stream, and the invert levels at the high and low end decided upon. Unless it is intended to have a diversion of the stream at either the high or the low end, the culvert will follow a direct course from a point in the bed of the stream near the toe of the slope of the embankment at the high side to the corresponding point on the low side. This centre line must be pegged out accurately, the pegs in most cases requiring lining-in with the theodolite. Levels should be taken on these centre pegs and the depth of excavation at each point calculated and marked on a label of wood, which is inserted next to the peg.

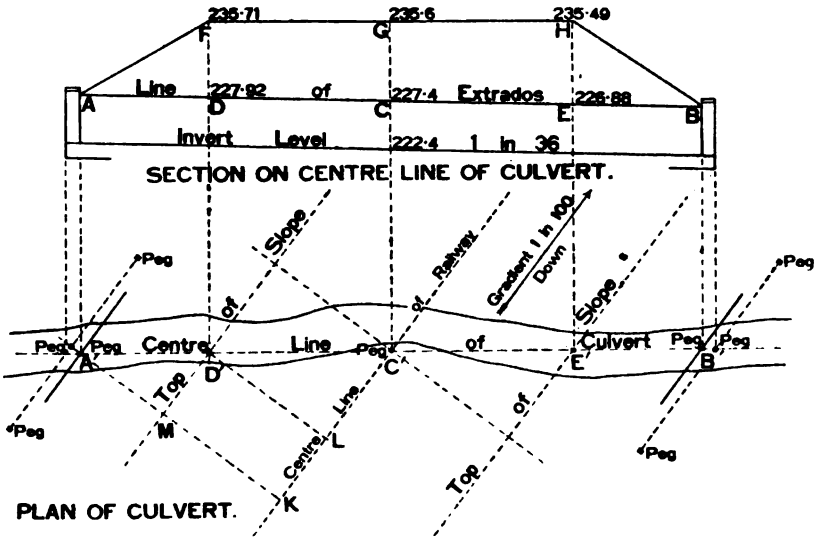

FIG. 73.

In fixing the face lines of the ends of the culvert the points where the two slopes of the embankment meet the extrados of the arch (or top of the crown) must be calculated, as here the slopes will finish against the parapet tops of the face walls, as at A and B in Fig. 73.

If the culvert is square to the centre line of railway the location of the points A and B, and therefore the faces, is a simple matter. But if the centre line of the culvert is oblique to the centre line of railway the operation may be by no means simple. If the railway is on a gradient and not level, the operation is still more complicated. Example in Fig. 73 will illustrate the method usually adopted.

The railway is on a down gradient of 1 in 100. The side slopes

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of the embankment are $1\frac{1}{2}$ to 1. The rate of fall of centre line of culvert is found to be 1 in 36, and the invert level at the centre line of the railway C′ is calculated to be 222·4. The level on the extrados at the crown is 5 feet above the invert level, consequently the level of C is 227·4. Again, on the plan the angle A′C′K between the centre lines of railway and culvert is measured and its tangent is equal to 1·33.

$$\therefore \frac{A'K}{KC'} = \frac{4}{3} \text{ and } \frac{AC}{A'K} = \frac{5}{4}$$

If the half-width at formation D′L is 15 feet, and the level at formation G is 235·6, the level at F will be the same as the top of the slope at D′ or L.

But $LC' : LD' = 3 : 4 \quad \therefore LC' = \frac{3}{4} \times 15 = 11{\cdot}25$ feet.

Correcting the level for the 1 in 100 gradient on this distance the level at F becomes 235·71, and similarly that at H 235·49.

Now the rate of slope (of FA) for the oblique direction D′A′ down the side slope, may be found from the data of triangle MD′A′, which has its sides $D'M : MA' : A'D' = 3 : 4 : 5$. Using these numbers as distances will enable the rate of slope in FA or D′A′ to be calculated.

Thus the gradient 1 in 100 makes D′ or F lower than M by $\frac{3}{100}$.

Again, the $1\frac{1}{2}$ to 1 slope makes A′ lower than M by $\frac{4}{1\frac{1}{2}}$. $\therefore$ D′ or F is higher than A′ or A by $\frac{4}{1\frac{1}{2}} - \frac{3}{100} = \frac{8}{3} - \frac{3}{100} = 2{\cdot}64$

$\therefore$ the rate of slope from D′ to A′ (or for FA) is—

$$\frac{5}{2{\cdot}64} = \frac{1{\cdot}89}{1} \text{ or about } 1\tfrac{8}{9} \text{ to } 1.$$

Now, knowing the rate of slope of FA and also that of DA (1 in 36 up), and the height DF 7·79 feet (the level at D is calculated for the distance D′C′ ($\frac{5}{4} \times 15$)), the distance D′A′ can be calculated.

$$\text{Thus 1 in } 1{\cdot}89 = \frac{1}{1{\cdot}89} \times 36 \text{ in } 36 = 19{\cdot}1 \text{ in } 36,$$

$$\text{and 1 in } 36 = \underline{\quad 1 \quad} \text{ ,,}$$

$$\therefore \text{ total effective slope is } 20{\cdot}1 \text{ ,,}$$

That is $FD : D'A' = 20{\cdot}1 : 36 = 7{\cdot}79 : D'A'$

$$\therefore D'A' = \frac{36}{20{\cdot}1} \times 7{\cdot}79 = 14{\cdot}0$$

But $D'C' = 15 \times \frac{5}{4} = 18{\cdot}75$

$\therefore$ total distance $C'A' = 18{\cdot}75 + 14{\cdot}0 = 32{\cdot}75'$ feet

Similarly the point B′ may be determined. The thickness of the face wall must be set off from A′ at right angles to the direction of

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railway and gives a point on the face of the culvert; two other pegs must now be fixed on the face line parallel to the centre line of railway at K. If the face is extended on both sides of the invert to form wings, the ends of these should be pegged out on the face line, and the excavation pegs put in by taking measurements from the face pegs. If the wings are parallel to the centre line of the culvert, their ends will be found by continuing the 1·89 to 1 slope from the crown A to its intersection with the surface.

Remarks.—The above method of setting out an oblique culvert could have been avoided if a complete survey and drawing of the culvert had been prepared, but the results obtained are more exact than could be gained from a working drawing, and in any case the accuracy of the drawing should be checked on the ground. It is always advisable in making the above calculations to make a sketch, not necessarily to scale, of the plan and section, and to enter on it the data as it is calculated. It can afterwards be used for preparing the drawing in the office from which the quantities can be taken.

The Preservation of "Setting Out" Marks

It is a good policy to impress all foremen with the importance of the "setting out" work. Nothing is more exasperating and wasteful of time than to have important centre pegs or other fixing points destroyed indiscriminately by gangs of workmen through the foreman not duly warning them of the importance of preserving them. Never pass over the destruction of a peg, even of little importance, without making the foreman feel very fully the seriousness of his crime. In this way only can that proper respect for "setting out" marks become imbued in the minds of all engaged upon the work of construction, and the need for frequent settings out of the same work be entirely removed.

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CHAPTER XII

PERMANENT WAY

Ever since the first railway was inaugurated the weights of engines and rolling stock have increased, and if the increases in speed, which have been more or less in proportion, are also taken into account, the need for corresponding increases in strength in the design of permanent way has occurred naturally from time to time in order to meet these contingencies.

It is the authors' intention here to deal only with present-day practice, but for those interested in the gradual development of railway construction there are several books dealing with historical facts on the subject.

English Practice.—The permanent way adopted almost universally in England is the double-headed type of steel rail resting in cast-iron chairs, held tight by means of wooden keys, the chairs being secured to transverse or cross wooden sleepers by screws, or a combination of screws and spikes.

The rails now used on the first-class lines are of British Standard Sections, the weight per linear yard varying from 90 lbs. to 103 lbs. or more. The following figures and diagrams show generally the principle adopted, although each Company's design differs in detail in respect of chairs, fastenings and distance apart of sleepers. Some Companies place the keys on the inside of the rails, and as the "running" does not appear to be affected, the rights of the matter seem to resolve themselves into one of opinion. It may, however, be interesting to state that on a railway where the keys were placed on the outside, the authors have on several occasions seen a train at reduced speed pass safely over three lengths of 30-foot rail, which had been merely "fished" with the plates at the ends, the platelayers not having time to insert a single key. Such a contingency, of course, occurs very rarely nowadays, but it is doubtful whether in similar circumstances the rail would have remained in position in the chairs, if the inside type of keying had been in operation.

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It is necessary for the young engineer to make himself familiar with the laws of the country governing the construction of all railways, but the following matter refers only to those having a gauge of 4 ft. 8½ ins. inside to inside, and 5 ft. 2 ins. outside to outside of rails.

For a double line 16 ft. 4 ins. is required from outside to outside of the outer rails. This allows a clear 6 feet between the inside rails. Again, for a quadruple line 42 ft. 8 ins. is required, which allows for a clear space of 10 feet between the middle rails. A siding should not be placed nearer than 10 feet to a running line.

It should be noted that, where necessary, it is permissible to place signals in the 10-foot spaces. Advantage is frequently taken of this, particularly where there is a large number of running lines approaching an important station.

Design.—In designing new or scheming the improvement of permanent way, where points and crossings are required, it is of the utmost importance properly to calculate and plot the whole arrangement

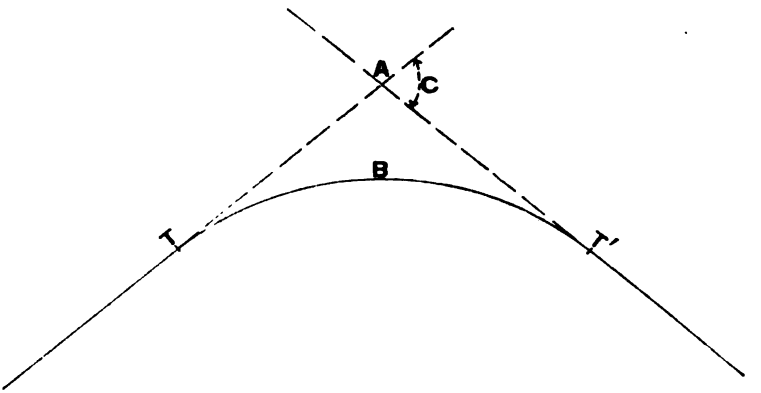

FIG. 74.

to a large scale, such as 8 or 10 feet to an inch. The 8-foot scale is the more reliable and useful, as it is possible to scale to inches on a carefully drawn plan. A few hours' careful work in the office will often save several days on the ground "setting out," not to mention the expense occasionally involved in getting the work to fit accurately.

To take a simple case, let it be required to introduce a curve between two straights as shown in Fig. 74. Here the straights must be continued beyond the intersection point A, and the angle C carefully measured with a protractor. If the tangent lengths AT and AT', which should be equal, are known, the radius of the curve can be obtained from the formula $R = AT \cot \frac{1}{2}C$. Again, if the distance

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AB is fixed, the radius of the curve and the tangents can be obtained. (For further data of circular curves, see Chapter VII.)

Again, let it be required to introduce a sharp curve between a flat curve and a straight (see Fig. 75). Select a point on the flat curve,

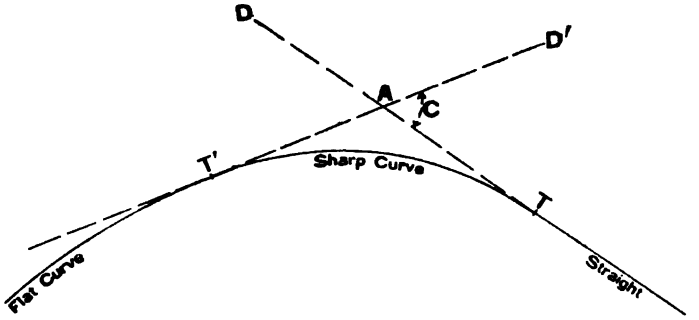

FIG. 75.

say T', and from it run a tangent line in the direction of D'. At the intersection A between this line and the straight measure the angle C, also the length AT', and calculate the radius of the curve by the formula given above. (The tangent length TA will, of course, be made the same as T'A.) If this radius is not found quite satisfactory, another point must be selected on the flat curve and the operation repeated until the desired improvement is effected.

The same principles, as above, are adopted in designing compound and reverse curves [*Note.*—Reverse curves must never be used on main lines, as vehicles travelling at a high speed tend to leave the road at the point of reverse curvature.] In the case of reverse curves a length of straight should be introduced between the two curves sufficient

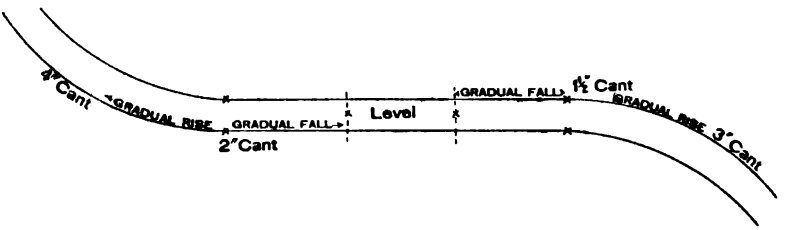

FIG. 76.

to admit of the full amount of superelevation being given to the rails at the tangent points. A good general practice, in the case where the length of straight is limited, is to provide, say, half the amount of cant which the curve requires at the tangent point, and run down into the straight at the rate of 1 inch in 60 feet, or in special cases 1½ inches in

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60 feet, until the level is reached. The length of level road should be at least equal to the longest vehicle likely to be used on the road, if good running is to be obtained. The length of straight in the case shown in Fig. 76 would be about 290 feet, assuming the allowance of a portion of level line 80 feet long.

Where curves are of less radius than 130 chains transition curves should be adopted (see Chapter VIII.).

Points Leading Off a Straight Road.—The method adopted in some text-books of continuing the curve of a connection right up

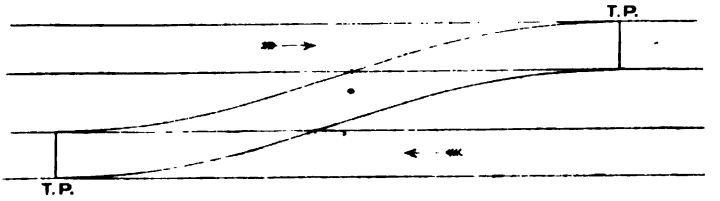

Fig. 77.

to the toe of the points is quite impracticable on the ground, as can be seen clearly by examining the detail of a typical switch shown in Figs. 83 and 86. All switches up to 20 feet in length are perfectly straight from toe to heel, and although switches much longer than 20 feet have been tried in order to curve the switch approaching the heel, they have not proved a success, in consequence of trouble arising from the sharpness of the blade at the toe. The blade quickly becomes worn and untrue and requires constant attention.

For a simple cross-over road with trailing points a convenient length of switch is 12 feet (Fig. 77).

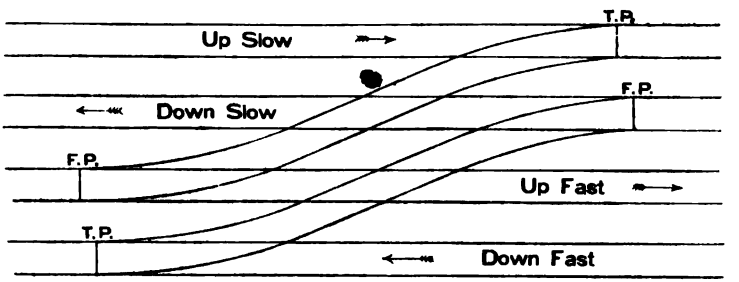

Fig. 78.

For a junction cross-over road from one running line to another running line in the same direction (see Fig. 78), the switches may

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be 16 feet long, and for a first-class junction (see Fig. 79) probably 20-foot switches would be adopted if the curves were not sharp.

The rails are usually represented on plans by a single line, and every plan should state by a note thereon, whether the gauge as plotted

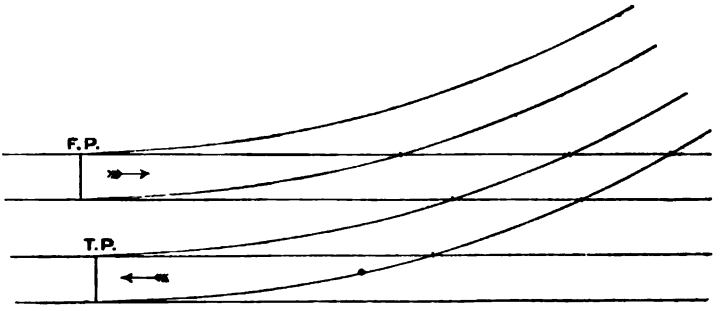

FIG. 79.

is to the insides of the rails, the centres of the rails or their outsides. The gauge adopted in this treatise is with reference to the outsides of the rails. This has been selected as the most convenient in practice as the result of many years' experience of surveying, designing, and "setting out." The point is dealt with more in detail in the section on "Surveying Permanent Way" (see p. 243).

The offset or throw at the heel of every straight switch is generally $4\frac{3}{4}$ inches, *i.e.* the distance from the outside edge of the heel of the

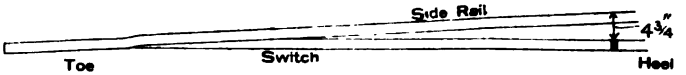

FIG. 80.

switch to the outside edge of the fellow rail adjoining the switch (see Fig. 80).

The length and position of the points having been decided upon, the standard offset referred to above of $4\frac{3}{4}$ inches at the heel should be

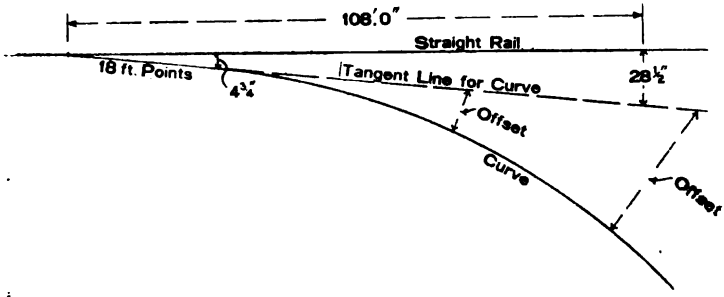

FIG. 81.

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pricked off on the drawing, and an indefinite straight line drawn with a fine line in pencil from the toe through this mark. This line gives the correct direction of the points, and is obviously the proper tangent line from which the new curve should be ranged by offsets in the usual way, commencing at the heel. As a means of obtaining more accurate plotting it is advisable to multiply the $4\frac{3}{4}$ inches by some convenient number, and the length of the points by the same number.

For example (see Fig. 81), take 18-foot switches and a throw of $4\frac{3}{4}$ inches, multiply this by say 6, and plot the product $28\frac{1}{2}$ inches at a distance along the straight rail equal to $18' \times 6 = 108$ feet measured from the toe of the points. This line gives more accurately the true direction of the switch, and is also the tangent line for the curve.

Points Connecting with the Outside of a Curve.—This case, as shown in Fig. 82, is somewhat more complicated than the last one,

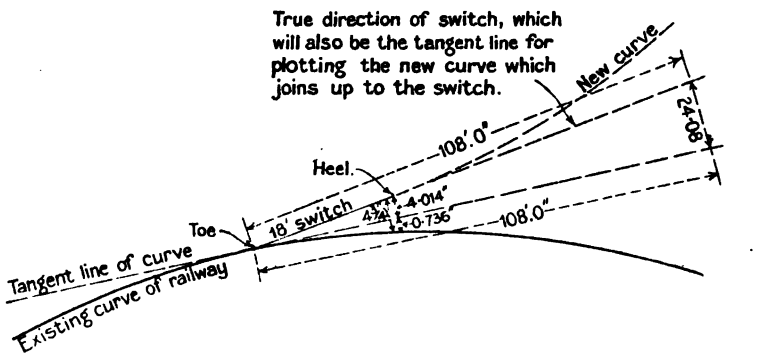

FIG. 82.

but provided the same principle is followed, there should be no difficulty in obtaining a perfectly correct plan.

The tangent line to the existing curve is drawn as shown in figure, and the offset from this tangent line to the curve opposite the heel of the points is calculated by the formula $O = \frac{L^2}{2R}$, where L is the distance in feet set off on the tangent line, R = radius in feet, and O = offset in feet. Table XXIV., p. 294, gives these offsets worked out for various lengths of points.

The standard distance from outside to outside of rails at the heel of the points being $4\frac{3}{4}$ inches, as previously stated, the offset distance as calculated by the formula must be deducted from this amount, and the remainder after being multiplied by some number (such as 6) should be plotted along the tangent line at a distance from the toe equal to the length of the points multiplied by the same number. For example, if

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18-foot points come off a curve of 40 chains radius the offset from the tangent line at the heel $= \frac{18 \text{ ft.} \times 18 \text{ ft.} \times 12}{2 \times 40 \text{ ch.} \times 66} = \cdot 736$ inches. To obtain the true direction of the points, this amount must first be deducted from the $4\frac{3}{4}$ inches, which leaves the remainder 4·014 inches. If this be multiplied by some number, say 6, the product 24·08 inches must be set off at a point along the tangent 18 ft. × 6 = 108 feet from the toe of the points. The line joining this point to the toe of the points is now considered the tangent line for the new curve which is required to join up tangentially to the heel of the points. This new curve is now plotted by offsets in the usual way.

Points Connecting with the Inside of a Curve.—Where, as shown in Fig. 83, the new connection comes off on the inside of a curve, the off-

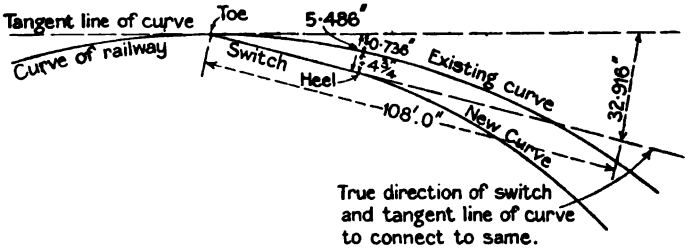

FIG. 83.

set ·736 inches must be added to the $4\frac{3}{4}$ inches giving 5·486 inches. Adopting 6 as multiplier as before gives 5·486 × 6 = 32·916 inches, and this amount as offset must be plotted from the tangent line of the curve at a distance 18 ft. × 6 = 108 feet from the toe of the points.

The whole procedure is quite simple, the only likelihood of error occurring where the ·736 inches is deducted from the $4\frac{3}{4}$ inches in the one case and added to it in the other. But a rough sketch will always remove any doubt on the point, and if an incorrect detail occurs, its presence can only be attributed to carelessness.[1]

Setting Out.—In setting out junctions on the ground, the same operation is adopted as that performed in the drawing office. A peg is driven into the ground to mark the position of the heel of the points, and a nail is driven into its top to denote the exact spot. If a theodolite is to be used it is set up over this nail and adjusted in the usual manner. As the peg which was inserted at 108 feet from the toe of the points gives the true direction of the tangent line, the instrument should be

[1] Table XXIV. will be found very useful for this work. The offset ·736 inches taken in above example is ·736 inches on a curve of 40 chains radius at a distance of 18 feet from the tangent point.

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sighted on the nail in this peg whilst the vernier reads zero. The angle for the first point on the curve is deflected from this direction, and the peg driven in on the line of sight and also at the correct distance from the tangent point and then the rest of the curve run in by pegs at every chain or, if in a complicated junction, at every half-chain.

If the drawing has been plotted to a large scale, the angles of all the crossings can be obtained by running tangents from the intersections, but it is advisable in all cases to set out the new connections on the ground by means of string lines which are held to the curve by steel pins or arrows of various lengths inserted in each sleeper or the ballast. The following instructions give the method of procedure :—

"Mark the rail with chalk showing the position of the new switches and insert an arrow at the heel $4\frac{3}{4}$ inches away to represent the throw. Set out the lead crossing as measured from the plan, and insert an arrow at the heel to comply with the angle found on the plan. If there are diamond crossings, etc., they should be set out first of all from the plan in the manner described, and afterwards the string lines may be run round the fixed pins, while intermediate pins can be driven for the purpose of lining out the curve to the eye, which after a little practice becomes quite easy. When this operation is completed, the string lines may show that by substituting another crossing, only very slightly larger or smaller as the case may be, a better-looking road can be obtained. There is no objection to such an amendment, and the angle shown on the drawing can be modified accordingly. It is surprising, however, in practice, how very rarely it is found necessary to deviate from the angles of crossings shown on the plan, provided due diligence has been exercised in the office and on the ground."

It is useful to mark the side of the existing rails with a cold set where the new points and crossings occur, and for the information of the Inspector.

The following marks may be used (see Fig. 84) :—A vertical mark

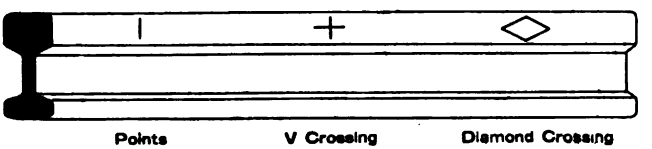

FIG. 84.

for points, a cross for V-crossing, and a sort of rhomboid for a diamond crossing.

In actual practice it is impossible to commence the curve exactly at the heel of the points in consequence of the fish-plate joint and the following adjoining sleeper, accordingly, for precise working on the

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drawing, about two feet should be added to the length of the points in order to obtain the effective length of straight before embarking on the new curve.

Leads.—The length of a "lead" is usually understood to be the length from the toe of the points to the splice of the V-crossing as shown in Fig. 85. In addition to the graphical method of finding the lead, etc.,

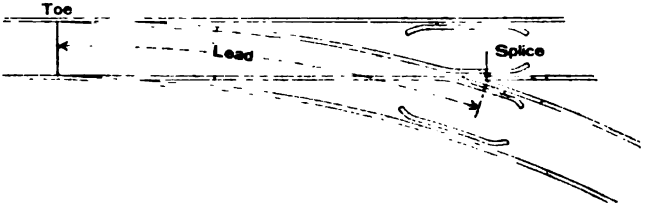

FIG. 85.

just described, there are also formulæ for calculating leads, but experience has proved that there is less likelihood of error if the connections are drawn out to a scale of not less than 8 feet to an inch, especially where they are at all complicated.

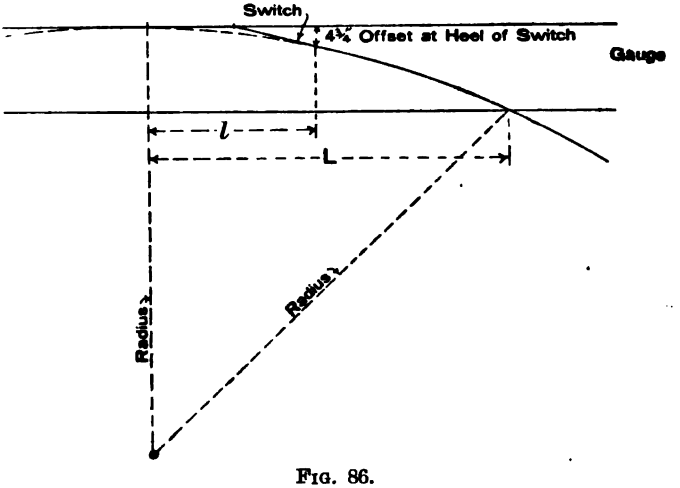

FIG. 86.

To determine the length of the lead.—The following formula is quite near enough for all practical purposes (see Fig. 86) and does not require the use of trigonometry :—

$$(1) \quad O = \frac{L^2}{2R}$$

$$\therefore L^2 = O \times 2R$$

$$\therefore L = \sqrt{(O \times 2R)}$$

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$$(2) \quad O_1 = \frac{l^2}{2R}$$

$$\therefore \ l^2 = O \times 2R$$

$$\therefore \ l = \sqrt{(O \times 2R)}$$

where O = offset or gauge
and O_1 = offset at heel of switch.

The length of the lead can therefore be seen to be L − l + length of points.

These formulæ are applicable only where a connection leaves a straight road, but in the case of a curve connection with a curve, the calculation is a very lengthy and tedious operation, and it will be found much more expeditious and accurate to draw the connection out to a large scale as previously suggested.

The diagram shown on Fig. 110 is generally sufficiently accurate for practical purposes, and a description of the basis of its compilation can be found on p. 233.

Notes on Practice.—The railway companies usually adopt their own particular standards in many details of permanent way design, but an analysis of these, which are many, would be more confusing than helpful. It is therefore decided, with the object of simplifying the matter, to consider only the designs of one of the leading English railways, as, except for slight differences of design in most cases, what applies to one applies to all.

For instance, the standards of all companies differ in respect of the spacing of sleepers, size and spacing of point and crossing timbers, size and weight of chairs, chair fastenings, fish-plates, and so on. But if it was possible to obtain a reliable comparison of each under the same conditions, there would be probably not a great deal of choice between them. It is a matter of fact that some tracks do not run as smoothly as others, but in all probability a superior kind of ballast is used in the latter case, and in addition greater expense incurred for labour in maintaining the road to a good level.

The following remarks refer to the type of permanent way used on the London and North Western Railway. These particulars with reference to placing points and crossings in particular positions, where the connections are close together, are given in order that both in planning and "setting out," the engineer may first make himself familiar with or have details at hand to refer to, the standards adopted by the railway he is engaged upon.

In Fig. 87 is given detail of a V-crossing. In these the distance from splice to heel may possibly vary in every angle of crossing from 1 in 2 to 1 in 12, and it is found that, frequently in complicated crossings, where slip roads and other connections have to be introduced, it is a great

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convenience to reduce the length of one of the sides of the V-crossing by cutting off a portion with a set or saw. But it is essential that the portion cut off is convenient in practice ; that is, it must be arranged in such a way that the joint comes between two timbers placed suitably for that particular crossing, and also that sufficient space remains between

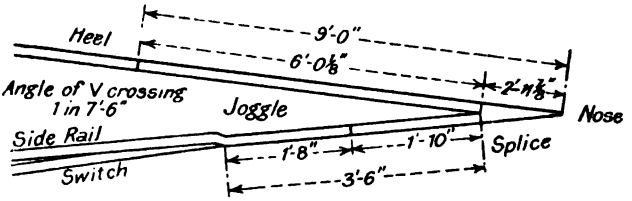

FIG. 87.

the legs of the crossing for properly tightening the bolts of the fish-plates. The shape, size and position of the blocks and chairs vary slightly with every angle of crossing, and the positions of the chairs determine the positions of the timbers.

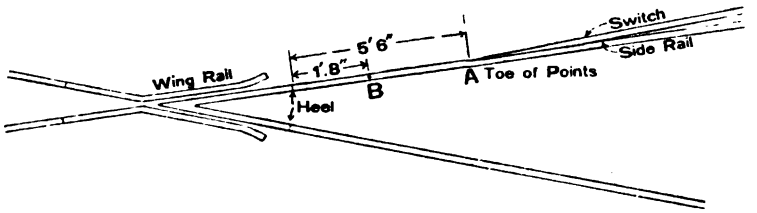

FIG. 88.

The standard distance from toe of points to "side rail" joint is 5 feet 6 inches (see upper connection on Fig. 88), and this is in accordance with the British Standard Specification. The most economical job is to fish the side rail as delivered to the heel of the crossing, but if, for some

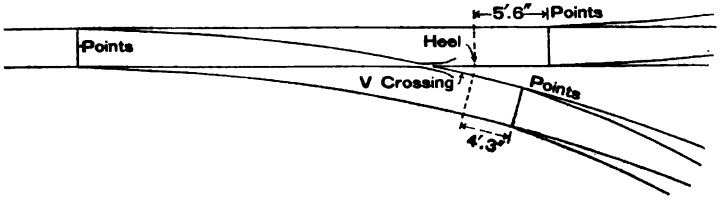

FIG. 89.

particular reason, it is possible to obtain a more suitable connection by placing the points nearer to the crossing, then the best system of timbering is obtained by cutting the side rail to enable the points to be placed at

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a distance of 1 foot 8 inches from the heel of the crossing, viz. at B instead of A in Fig. 88.

The arrangement shown in Fig. 89 is frequently met with, where two switches are required to be placed near the heel of a crossing, and it is not good practice to place the two pairs of switches at an equal distance from the heel of the crossing owing to the difficulty of timbering and point rodding. A much better road will be obtained by fixing one pair of points to the standard of 5 feet 6 inches from the heel of the crossing and the other pair 4 feet 3 inches as shown in Fig. 89.

Diamond Crossing.—A typical detail of diamond crossing is illustrated in Figs. 90 and 91, the latter representing an angle of 1 in 8,

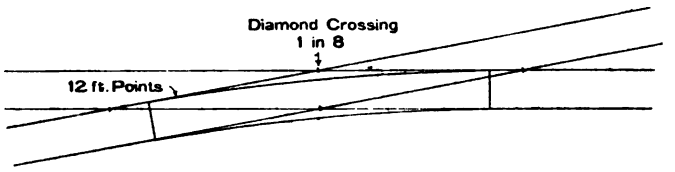

FIG. 90.—Single slip.

which is the flattest angle which the English Government Regulations will permit. On the other hand, it is permissible to use diamond crossing as wide as desired down to 1 in 1 or even to where one road crosses another at right angles. Such wide crossings, however, should be avoided as far as possible, as they are most troublesome to maintain satisfactorily. Another objection is that when the wheel passes from the wing rail to the nose of the V-crossing—a space of $1\frac{3}{4}$ inches, a very severe jolt is felt in the train and this constant hammering soon causes the crossings to wear.

When designing a single-slip or double-slip road, as shown in Figs. 90 and 91, it is essential to adopt a diamond crossing of either 1 foot in

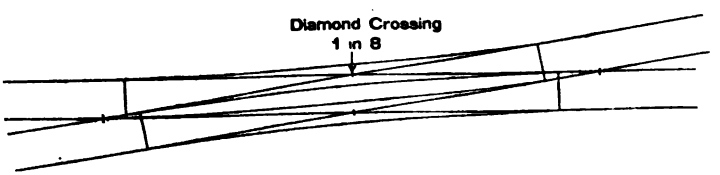

FIG. 91.—Double slip.

7 feet 6 inches or 1 in 8 to ensure that a suitable curve may be obtained between the switches at each end of the slip. It is evident that the sharper the intersection of the two roads is made the nearer together become the V-crossings at each end, with a consequent sharpening of the slip-road curve.

Continuing the same reasoning, the length of switches should be

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almost the shortest, say 12 feet, to obtain the greater "throw" already explained in the preceding pages.

Junction Cross-road.—In a junction cross-road from one running line to another (see Fig. 92,) it will be found convenient first to draw

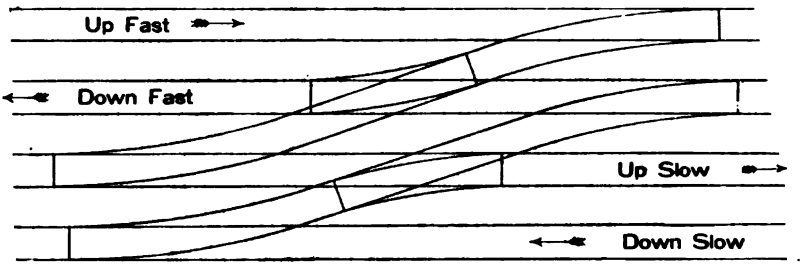

FIG. 92.—Junction cross-roads from one running line to another, with slip-road introduced to form simple cross-over road.

straight lines intersecting at an angle of 1 in 8 to represent the diamond crossing and then proceed to lay out a suitable line to join tangentially to the switches. As all curves below 10 chains radius in a running line are required by the Government Regulations to have a check rail, it is advisable to design a curve at least 12 chains radius in order to make sure that in the actual work no part of the "lead" will be less than 10 chains when checked by versines. Switches about 16 feet long should

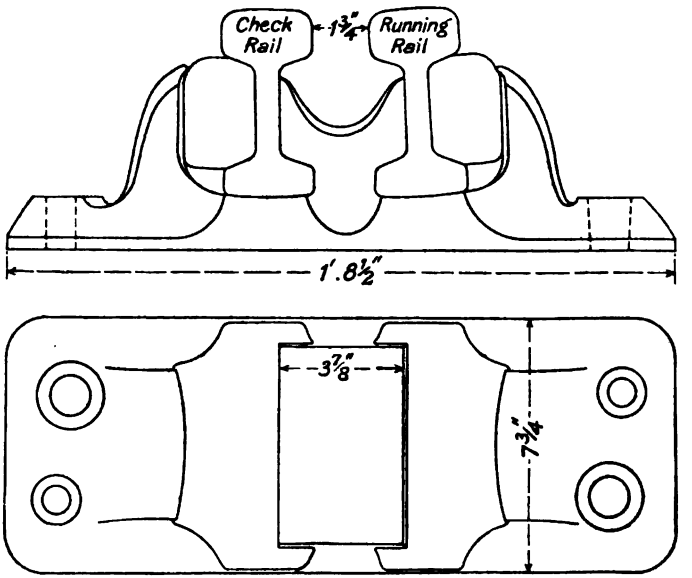

FIG. 93.—Check chair.

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be selected for these junction cross-roads, where a train passes through on its direct journey.

Slip Road.—Occasionally it is convenient to introduce a slip road for shunting purposes, thus obtaining a simple cross-over road as shown included in Fig. 92. The check rail is placed $1\frac{3}{4}$ inches away on the inside of the lower running rail as shown on Fig. 93, and it is introduced for the purpose of securing greater safety to passenger trains when passing round a sharp curve, for the reason that ordinarily the flanges of the wheels have a tendency to ride the outer rail owing to the centrifugal force. The check rail adjoining the lower rail is placed near enough to prevent this riding tendency, as is proved by the worn appearance of

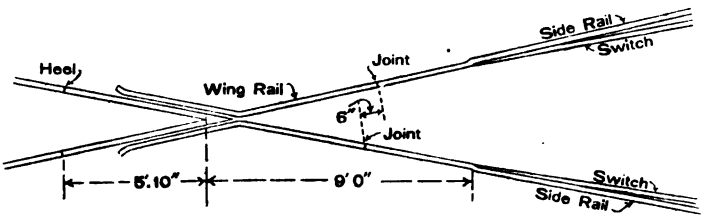

FIG. 94.

the inside edge of the check rail due to the rubbing which it receives from contact with the flanges of the wheels.

A practical issue worth mentioning arises out of the designing of a double slip road. The desirability of placing the switches at each end as far away from one another as permissible has been established and a little help in this direction may be obtained by cutting a piece off the wing rails, as shown in Fig. 94. For good timbering one pair of points should be 6 inches in advance of the other pair where both are trailing points. Should the points, however, be facing, then the distance between them should be 2 feet 8 inches instead of 6 inches to allow for the introduction of the signal apparatus or locking bar, etc.

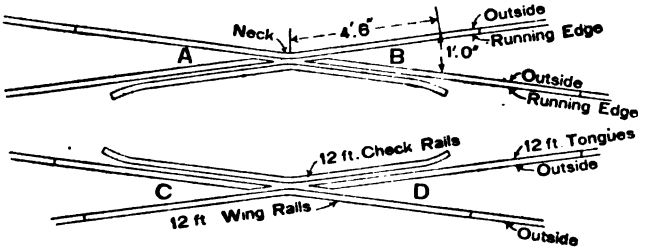

FIG. 95.—Diamond crossing, 1 in 4' 6".

Adjusting Diamond Crossings.—Fig. 95 gives a detail of what may be termed a "fixed" diamond, to distinguish this type from the

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more modern form, with movable tongues controlled from the signal cabin, which latter is described later.

In the fixed diamond crossing there must be a rather wide space left in the centre, or neck of the diamond, to permit of the flanges of the wheels travelling across, because it is in the neck of diamond crossings where a great many wagons leave the road during shunting operations. It is quite obvious that, if the wheels of a vehicle come to a stand in the neck, especially on a curved road, and the train be suddenly shunted, as all goods trains usually are, these particular wheels are as likely to take the wrong road as the right, thus causing a derailment. Hence there is some justification for the rule, which is not always observed, that shunters shall not bring a goods train to a stand over a diamond crossing on account of the risk of derailment.

Fig. 96 is an exaggerated sketch of a diamond crossing, which has either not a suitable angle for the position, or is out of adjustment.

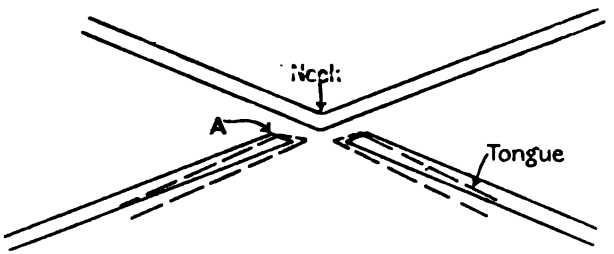

Fig. 96.

When in a temporary difficulty, it is well to remember that it is safer to make a flat diamond sharp, than a sharp one flat. If the corner A, for instance, in a sharp diamond is closed in for the purpose of making the crossing flatter, then knocking will take place during the passage of vehicles.

"Movable" or "Switch" Diamond Crossings.—In Fig. 97 is shown an example in plan of a movable diamond crossing with switches 16 feet long.

Switch diamond crossings have been used extensively in America, and also on many of the Colonial railways, but in England there has been not a little hesitancy in the application of this type of crossing, even in situations where they would have been most useful and convenient.

There is no question whatever regarding the success of those crossings from a purely permanent way point of view, as a solid and continuous bearing is given to the running rail, thus reducing in large measure the hammering of permanent way and rolling stock, which is a matter deserving the greatest attention.

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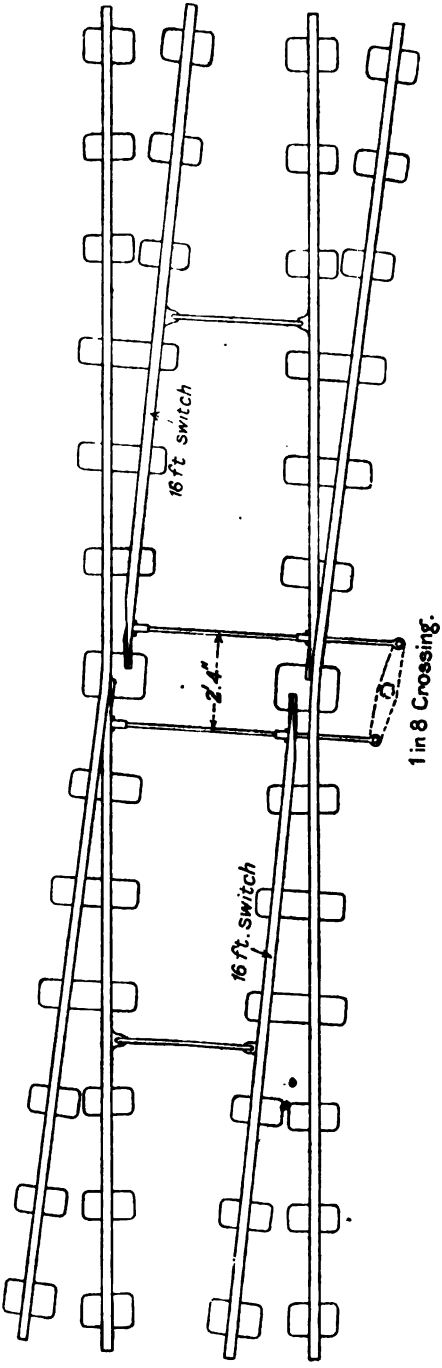

FIG. 97.

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There are other advantages which enable the Track Engineer to clear up difficult problems. Of these, not the least is that practically any angle of crossing may be used where switch diamonds are adopted.

It has already been stated that the flattest angle which the Board of Trade will permit in a fixed pattern diamond crossing is 1 in 8, but the Government inspectors have expressed themselves very strongly in favour of switch diamonds, and go so far as to approve of practically any workable angle.

Switch diamond crossings should be worked from signal cabins, and be bolted and detected in precisely the same manner as Facing Points. This requirement is considered by some engineers to be a disadvantage, or indeed a source of possible danger; others, again, believe that delays to traffic arise, and both put forward the increased expenditure in maintenance for the signal department. In spite of these expressions of opinion, there is not the slightest doubt that switch diamond crossings have been a great success, where they have been substituted for fixed diamonds in positions where derailments were formerly very frequent.

If properly and carefully laid the movable diamond provides a good and safe running road, and although the extra expense of connections to signal cabins, etc., may be rather heavy, there is a safety and comfort of running which cannot be estimated in £ *s. d.*

Altering Crossings.—In Fig. 87 is shown the nearest possible position in which a switch can be fixed to the splice of 1 in 7′ 6″ V-crossing. This, however, is one which would only be resorted to in very extreme cases, as the strength and efficiency is reduced proportionately to the cutting of the legs. It is preferable to have the shorter leg of the crossing 3 feet 9 inches instead of 1 foot 10 inches; but, as already stated, it is better from a maintenance point of view, and also cheaper, to design the connections so that the standard lengths may be "fished" together without cutting.

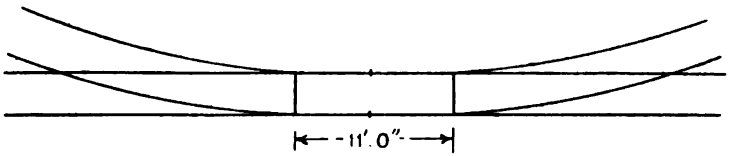

FIG. 98.

Switches.—In Fig. 98 is shown the standard distance between two pairs of points, namely 11 feet, but this distance may be reduced where necessary to 3 feet 4 inches by cutting to side rails.

If the straight road in figure had been a running line, then, it is evident, one pair of points would be "facing" points. In such a case

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they cannot be nearer together than 41 feet, in order to allow a sufficient length for the locking bar, as required by Government Regulations.

In Fig. 99 is given an example of a "three-throw," a type of siding connection largely used at one time, especially where space was a con-

FIG. 99.—Three-throw points.

sideration. The great objection, however, to this arrangement is that in order to get in the second pair of points it is necessary to have the road wider than the gauge by $1\frac{1}{2}$ inches, *i.e.* the gauge must be 4 feet 10 inches instead of 4 feet $8\frac{1}{2}$ inches, and this is a source of danger. "Three-throws" also require a great deal of attention to keep them in a proper state of repair. For these, and the foregoing reasons, they have been gradually replaced by the arrangement shown in Fig. 100, which is termed a "Pannier" lead.

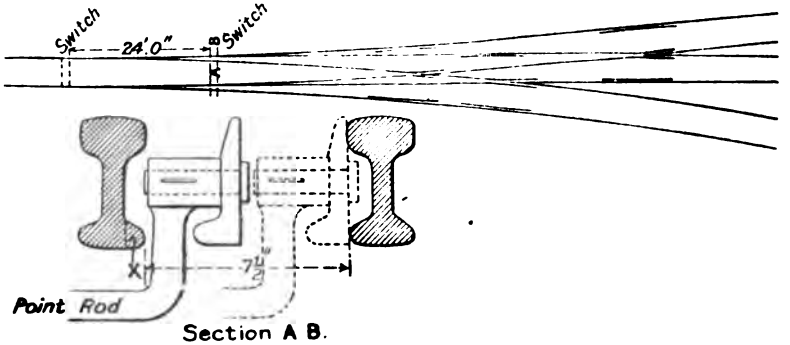

FIG. 100.- Pannier lead.

It can be seen by inspecting the detailed cross-section, that owing to point-rod connections there must be a clear space of at least $7\frac{1}{2}$ inches, where the second pair of points is situated, in order to permit of the points sliding backwards and forwards. This spot can be located by a simple calculation. Generally speaking, however, the second or following pair of points will be about 24 feet from the first pair, but even this distance can be reduced under exceptional circumstances by clipping a piece out of the lower flange of the rail, as at X on the section, whereby the space between the rails may be $6\frac{3}{4}$ inches. This alternative cannot be recommended for general practice, as the strength of the rail is

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reduced very considerably thereby, and frequent cases of broken rails are recorded, where rails have been clipped in this manner.

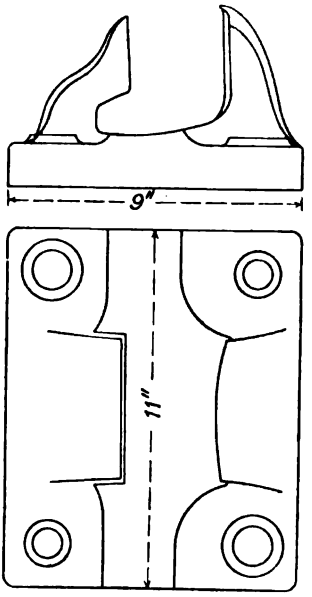

FIG. 101.—Bridge chair.

Special Chairs.—It has been mentioned previously that the position and shape of chairs vary with every angle of crossing. But occasionally it is advisable to resort to strategy, and depart from the use of specified chairs in situations where connections are close together, such as "pannier" leads, scissors crossings, etc. Sufficient space must always be provided between the timbers to permit of the platelayers inserting their picks and spades for removing the ballast, before they proceed to repair the road. If the timbers in a pannier lead were placed, in some instances, exactly to standard, the whole connection would practically be one mass of timber. To avoid this, it is usual, instead of interlacing smaller timbers, to adopt "through"

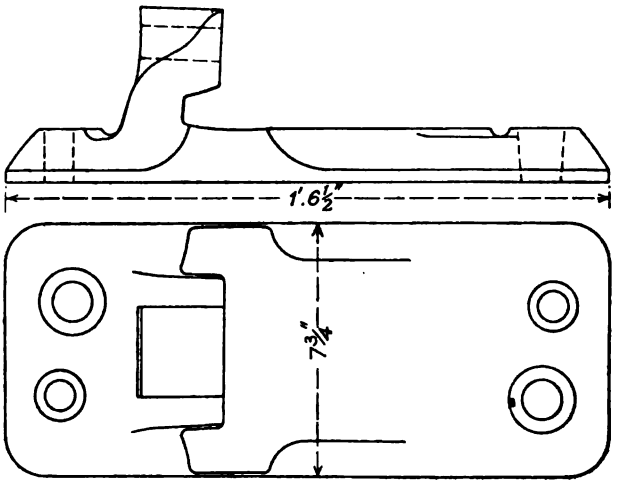

FIG. 102.—Point chair.

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timbers at convenient distances apart, moving some of the chairs forward and others backward in order to obtain the desired result. The positions of the chairs of a particular crossing do not admit of very much longitudinal movement, consequently in a case where the chairs, say, require to be placed nearer to the splice, for good timbering, it is possible to obtain a good result by introducing a similar chair belonging to a crossing with an angle half a size flatter. In some cases the special crossing chair is left out altogether, and bridge chairs and slide chairs (see Figs. 101 and 102) substituted.

Various methods of overcoming small difficulties like these suggest themselves to the engineer as each one arises, and it is not intended here to exhaust them, but rather to point out possible lines upon which to proceed when faced with difficulties of design, so that the lay out may be schemed in the drawing office to work in harmony with the fitting together on the ground.

Templates.—Wooden templates are sometimes made for crossings which have to be specially forged for the purpose, such as fiddle-back crossings, which are sometimes necessary in a scissors crossing (see Fig. 103).

On a perfectly flat piece of ground, or, better still, a large "setting out" platform built of wood, it is advisable to set out the connections with string lines, very accurately, and to take the templates and other particulars from these.

Although all this information could be obtained from the large scale drawings, it is essential to set out full size for all crossings which are to be specially forged, because, unlike the common V-crossing, it is impossible to slue them even to the smallest extent, when being fitted into the road without affecting the adjoining roads.

Special Crossing Timbers.—In a junction where both the main lines and branch lines are curved and require different cants, it is excellent practice to have special timbers made for the purpose. An example of this is shown in Fig. 125.

It is not necessary to prepare special timbers for the whole of the junction, those situated at or about the crossings being the most important, and the ordinary timbers in the lead may be adzed to a certain extent for varying the cants.

Timbers 12 inches wide and 6 inches thick or thereabouts should always be placed under the points and the crossings in order to give the necessary strength where required, and although a special timber lead throughout is preferable, ordinary sleepers are occasionally introduced in the lead between the point timbers and the crossing timbers. These sleepers must be interlaced, and therefore as practically double

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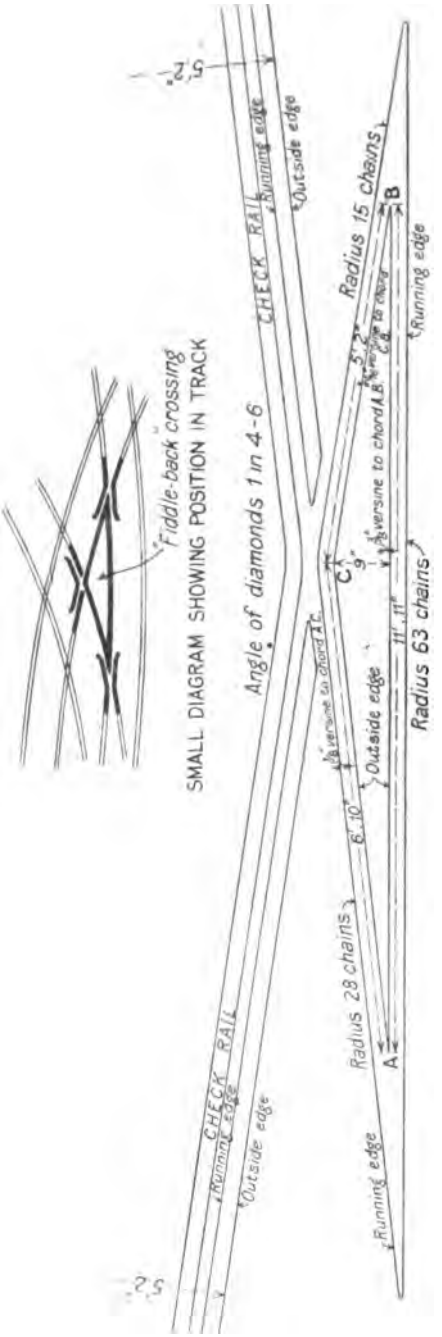

Fig. 103.—Fiddle-back crossing.

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the number of them is required, it is not very easy to pack the sleepers properly, although the difference of cant on the two roads can more readily be obtained.

Measuring the Angles of Crossings.—When new crossings are delivered on to the ground the angle of each one is usually marked in some distinctive manner for the convenience of those in command. The wing rail is usually selected for this marking, but it frequently becomes obliterated after some years in the road.

Fig. 117 shows a sketch taken from a field-book of an actual survey of a length of railway on which it may be of interest to note, that the angle of every crossing included in the survey is quoted. It is a very simple matter to find the angle, and if all measurements are taken to the outsides of the rails, as already recommended, and also if the width of the head of the rail is $2\frac{3}{4}$ inches, it will be seen that the true position of the splice of the V-crossing is at the point where the two outer edges of the rails meet. The exact point of intersection is where the crossing measures $2\frac{3}{4}$ ins. $\times$ 2 = $5\frac{1}{2}$ ins., and this point should be marked with chalk in the first place. Then if a pocket rule 1 foot in length be placed between the legs of the V-crossing, as shown in Fig. 104, the

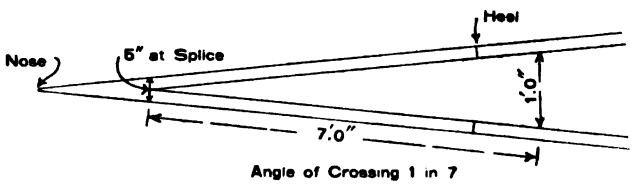

FIG. 104.

distance from this point to the chalk mark referred to above will give the angle of crossing required.

In the case of a flat crossing it will be sufficiently accurate to measure to a point where the legs are, say, 6 inches apart, and double this distance to the splice will then give the angle of the crossing. The reason for this is that in flat crossings the space of 1 foot occurs at a point so far beyond the heel of the crossing, that the ordinary rails joining up the heels of the crossing may be curved, thereby giving an erroneous result.

Diamond Crossing 1 *in* 4 *ft.* 6 *ins.*—In measuring the angle of a diamond crossing, it is most important to remember to place the pocket rule either from outside to outside of rails or from running edge to running edge. The former is recommended for the sake of consistency. In Fig. 95 is shown the 1-foot space at a distance of 4 ft. 6 ins. from the neck of the diamond, therefore the angle of the crossing is 1 in 4 ft. 6 ins.

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In practice it is essential to measure all the four legs as at A, B, C and D, and take a mean of the four in order to arrive at the correct angle.

V-crossings are usually to be obtained in the following standard angles :—

Between	1 in	1′ 6″	and	1 in	6′ 0″	they are made to every	3″	difference of angle
„	1 in	6′ 0″	„	1 in	11′ 0″	„ „	6″	„ „
„	1 in	11′ 0″	„	1 in	20′ 0″	„ „	12″	„ „

That is, any of the following may be ordered : 1 in 4′ 0″, 1 in 4′ 3″, 1 in 4′ 6″, 1 in 4′ 9″ and so on. The next range starts at 1 in 6′ 0″, and continues 1 in 6′ 6″, 1 in 7′ 0″ and so on, and finally 1 in 11′ 0″, 1 in 12′ 0″, 1 in 13′ 0″, etc.

The Ordering of Crossings.—When carefully extending the tangents on the detailed drawing for the purpose of finding the angle of crossing, it frequently occurs that a quarter-sized angle, say, 1 in 6 ft. 3 ins., which from the above list does not appear to be stock size, occurs, and the question arises as to whether a 1 in 6 ft. 0 ins. or a 1 in 6 ft. 6 ins. should be ordered. In most cases it is preferable to open the leg of the crossing rather than close it, and therefore in this case select the 1 in 6 ft. 6 ins., which when slightly opened on the job, will give a 1 in 6 ft. 3 ins.

Special cases do occur where the crossing might be preferably closed in slightly at the heel, as at A in Fig. 105, in which case a 1 in 6 ft.

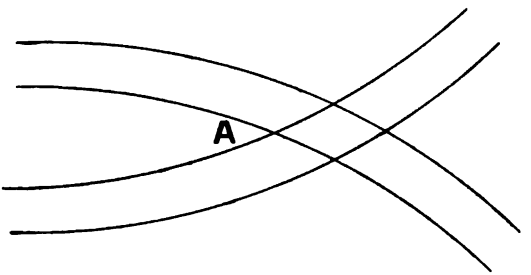

Fig. 105.

crossing would be ordered in preference, and the legs closed in with a Jim-crow in order to obtain the required 1 in 6 ft. 3 ins.

Sharpest Curves for Sidings.—In designing connections leading to sidings where engines are required to enter, it may be necessary to know the sharpest curve over which an engine will pass safely. A curve of 6 chains radius should be taken the absolute limit in this direction. In planning such connections it is always advisable to design to a 7-chains radius, because it is practically impossible to produce on the ground a curve consistently of 6 chains radius throughout.

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Carefully taken versines will expose in places a slightly smaller radius, and it will be found advantageous in cramped positions, such as these, to lay out the permanent way to a slightly increased gauge, such as 4 ft. 9½ ins. instead of 4 ft. 8½ ins. in order to enable the vehicles to pass round more easily. Instances have occurred where the chair fastenings have been actually drawn out of the sleepers, thus proving that a wider gauge is necessary in the very sharp curves which occur in sidings.

In sidings where engines are not required to enter, the curves may be as sharp as 1½ chains radius, and if the gauge is increased by 1 inch, ordinary wagons will pass safely round a curve of 1 chain radius.

Fish-plates.—In Fig. 106 is shown an elevation of what is known as a "cramped" or "joggled" fish-plate. The need for such a plate at this point is apparent, owing to the difference in size of the two rails which are joined together. This, which is liable to occur, is an instance of a new rail of heavier section being joined up to an old rail of lighter section.

Scissors Crossings.—Figs. 107 and 108 show alternative arrangements of two cross-over roads combined in such a manner as to form what railway engineers call a "scissors" crossing. The object of such crossings is to conserve the standing accommodation for vehicles, especially in railway stations; they are also used on account of their convenience.

These scissors crossings are usually installed in the stations of important junctions, and their usefulness may be understood to some extent by the following: Assume a train, made up of carriages for various destinations, to have been drawn into No. 2 Platform, and that another train follows into Platform No. 3, and which is bound, say, for London. Now all carriages labelled for London, which are attached to the train in No. 2 Platform, will be uncoupled, and a small tank engine, which in the meantime has come up in the rear, will draw the London carriages backwards, and then shunt them forward through the cross-over road to be attached to the main line train. In like manner, carriages which are attached to the rear of the London train, but which are intended for some other destination, will be detached and shunted back or through by a similar operation. In fact there are many ways in which these crossings are particularly useful for purely traffic purposes, but it is not intended to deal with these except for the purpose of introducing some engineering details.

When the adjacent tracks are spaced at a distance apart of exactly 6 feet, it will be observed from Fig. 107 that the special crossings at AA appear to be troublesome. Further, with only a 6-foot space between the middle rails it is almost impossible to obtain a really satisfactory

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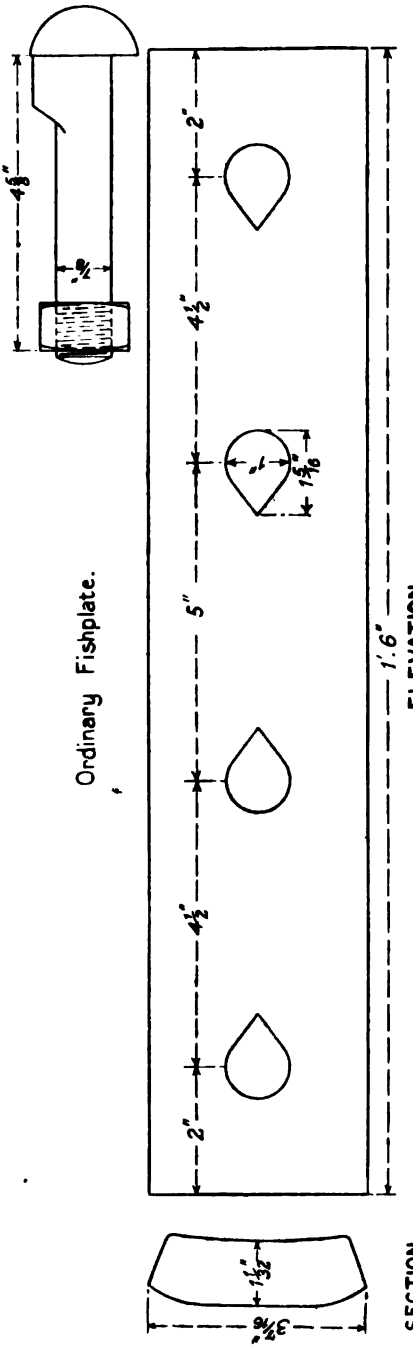
Ordinary Fishplate.
ELEVATION.
SECTION.
Smaller section of rail.
Elevation showing two rails of different sections joined together by means of a cramped or joggled fishplate.

FIG. 106.

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road. This particular design, therefore, should only be adopted where the middle rails are considerably more than 6 feet apart.

However, even where the middle rails are only 6 feet apart, the difficulty can be got over by adopting the system shown in Fig. 108.

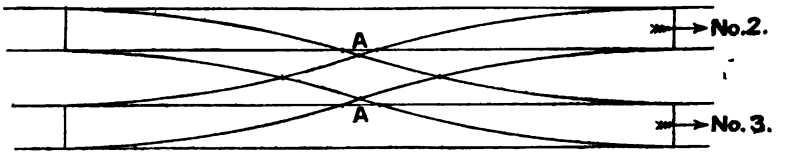

FIG. 107.

Much depends upon the radii of the curves in the leads, but if, as a preliminary, the points B and C are from 15 to 20 feet apart, and the curves arranged, by trial, in such a manner as to render the position of the crossings convenient from a practical point of view, the advantages will be obvious.

By cutting the legs and wing rails of the V-crossings, and possibly the tongues of the diamond crossing as shown on p. 217, many difficulties can be overcome, and the great convenience of shunting within a limited area probably be attained.

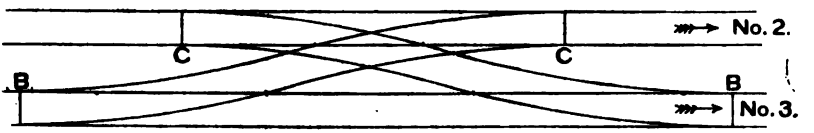

FIG. 108.

It is possible to get a good idea of the practicability or otherwise of a scheme of this character by drawing the rails, crossings, wings and check rails by double lines on plan to represent the head of the rail. If there is not sufficient clearance, when drawn carefully on paper, for the fitting together of the several parts it is more than likely that the string lines will not show a more satisfactory result. The problem is a very interesting one to work out, and the method shown in Fig. 108 is a solution.

Length of Outer Rails on Curves.—Table XIV. shows the increased length of the outer rail over that of the inner rail, on curves of various radii, both for rails of 60-foot and 30-foot lengths.

The practice of placing the fish-plated butt joints opposite to one another has survived the many so-called improved joints, which have been tested from time to time, and as it is desirable in this type to keep the joints as square with one another as possible, the table will be found very useful when ordering material for new lines or relaying.

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TABLE

TABLE SHOWING THE EXTRA LENGTH REQUIRED

60 feet rails.

Chains radius.	Length of outer rail.	Chains radius.	Length of outer rail.	Chains radius.	Length of outer rail.
6		35	60·1 9/16	78	60·0 11/16
7		36	60·1 1/2	80	60·0 5/8
8		37	60·1 7/16	85	60·0 5/8
9		38	60·1 7/16	90	60·0 9/16
10		39	60·1 3/8	95	60·0 9/16
11	60·4 13/16	40	60·1 3/8	100	60·0 1/2
12	60·4 7/16	41	60·1 5/16	105	60·0 1/2
13	60·4 1/8	42	60·1 5/16	110	60·0 1/2
14	60·3 7/8	43	60·1 1/4	115	60·0 7/16
15	60·3 5/8	44	60·1 1/4	120	60·0 7/16
16	60·3 3/8	45	60·1 3/16	125	60·0 7/16
17	60·3 3/16	46	60·1 3/16	130	60·0 3/8
18	60·3	47	60·1 1/8	135	60·0 3/8
19	60·2 7/8	48	60·1 1/8	140	60·0 3/8
20	60·2 11/16	49	60·1 1/8	145	60·0 3/8
21	60·2 9/16	50	60·1 1/16	150	60·0 3/8
22	60·2 7/16	52	60·1 1/16	160	60·0 5/16
23	60·2 3/8	54	60·1	170	60·0 5/16
24	60·2 1/4	56	60·0 15/16	180	60·0 5/16
25	60·2 3/16	58	60·0 15/16	190	60·0 1/4
26	60·2 1/8	60	60·0 7/8	200	60·0 1/4
27	60·2	62	60·0 7/8	220	60·0 1/4
28	60·1 15/16	64	60·0 13/16	240	60·0 3/16
29	60·1 7/8	66	60·0 13/16	260	60·0 3/16
30	60·1 13/16	68	60·0 13/16	280	60·0 3/16
31	60·1 3/4	70	60·0 3/4	300	60·0 3/16
32	60·1 11/16	72	60·0 3/4	350	60·0 1/8
33	60·1 5/8	74	60·0 11/16	400	60·0 1/8
34	60·1 9/16	76	60·0 11/16		

NOTE.—The lengths given are to the nearest 1/16".

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XIV.

FOR THE OUTER RAIL OF CURVES, TO KEEP THE JOINTS SQUARE.

30 feet rails.

Chains radius.	Length of outer rail.	Chains radius.	Length of outer rail.	Chains radius.	Length of outer rail.
6	30·4 2/16	35	30·0 3/4	78	30·0 3/8
7	30 3 5/16	36	30·0 3/4	80	30·0 5/16
8	30·3 7/16	37	30·0 3/4	85	30·0 5/16
9	30·3	38	30·0 11/16	90	30·0 5/16
10	30 2 3/4	39	30·0 11/16	95	30 0 5/16
11	30·2 1/2	40	30·0 11/16	100	30·0 1/4
12	30·2 1/4	41	30·0 11/16	105	30·0 1/4
13	30·2 1/8	42	30·0 5/8	110	30·0 1/4
14	30·2	43	30·0 5/8	115	30·0 1/4
15	30·1 13/16	44	30·0 5/8	120	30 0 1/4
16	30·1 11/16	45	30·0 5/8	125	30·0 1/4
17	30·1 5/8	46	30·0 9/16	130	30·0 3/16
18	30·1 1/2	47	30·0 9/16	135	30·0 3/16
19	30·1 7/16	48	30·0 9/16	140	30·0 3/16
20	30·1 3/8	49	30·0 9/16	145	30·0 3/16
21	30·1 5/16	50	30·0 9/16	150	30·0 3/16
22	30·1 1/4	52	30·0 1/2	160	30·0 3/16
23	30·1 3/16	54	30·0 1/2	170	30·0 3/16
24	30·1 1/8	56	30·0 1/2	180	30·0 7/16
25	30·1 1/16	58	30·0 1/2	190	30·0 1/8
26	30·1 1/16	60	30·0 7/16	200	30·0 1/8
27	30·1	62	30·0 7/16	220	30·0 1/8
28	30·1	64	30·0 7/16	240	30·0 1/8
29	30·0 15/16	66	30·0 7/16	260	30·0 1/8
30	30·0 15/16	68	30·0 7/16	280	30·0 1/8
31	30·0 7/8	70	30·0 3/8	300	30·0 1/16
32	30·0 7/8	72	30·0 3/8	350	30·0 1/16
33	30·0 13/16	74	30·0 3/8	400	30·0 1/16
34	30·0 13/16	76	30·0 3/8		

PERROTT AND PADGEB.

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Analytical and Graphical Computation of Leads, etc.

As has already been shown in the section on Permanent Way Design, p. 208, a junction cannot be truly tangential with the Main Line in consequence of the switch being straight, but a simple formula can be obtained from the assumption that the junction curve is tangential to a line running parallel to the centre line of the Main Line, and only a few inches away from it. This distance or deviation of the centre line of

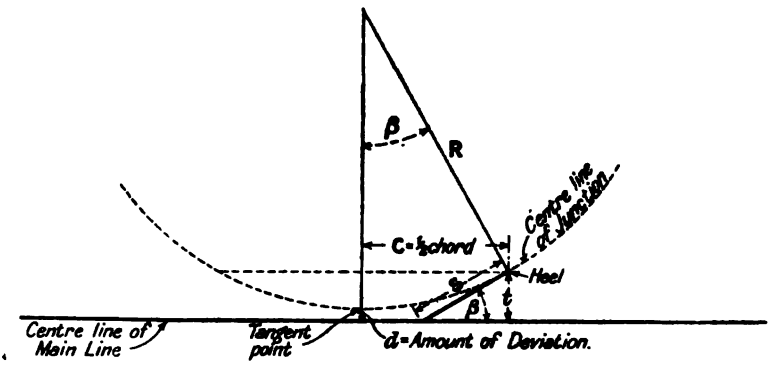

Fig. 109.

the junction from the centre line of the Main Line can be determined from the following formula (see Fig. 109) :—

Let t = throw or clearance at heel of switch (usually $4\frac{3}{4}$ inches).
s = length of switch to be used.
R = radius of centre line of junction.
C = length of half chord (tangent point to heel of switch).
V = versine to ditto.
d = amount of deviation $t - V$.
(All dimensions in feet.)

then $$\frac{C}{R} = \frac{t}{s} \text{ therefore } C = \frac{Rt}{s} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

But $V = \frac{C^2}{2R}$ and by substituting the value of C from (1) we get—

$$\frac{\left(\frac{Rt}{s}\right)^2}{2R} = \frac{R}{2}\left(\frac{t}{s}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$\therefore d = t - \frac{R}{2}\left(\frac{t}{s}\right)^2$$

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If R = radius in chains

then $V = 33R\left(\frac{t}{s}\right)^2$

and $d = t - 33R\left(\frac{t}{s}\right)^2$ ft.

This formula is the basis for producing the graph on Fig. 110, giving all particulars as to crossings, etc., incidental to a truly circular curve from the heel of the switch.

Explanation of the uses of Figure 110.—The lead is assumed to be the distance from the toe of the switch to the splice of the crossing, and although two persons may disagree slightly upon the reading of the figure, when it comes to probably a question of inches, still, if the particulars are carefully and judiciously taken from it, they will be found quite satisfactory for all practical purposes.

For simplification, centre lines have been adopted in calculating the graph, and it is evident that if, say, the centre line of the six-foot of a junction of 10 chains radius joins up to the centre line of the six-foot of a straight Main Line, the radius of the centre line of the four-foot way of the inner track will be less than 10 chains by 3 ft. + 2 ft. 7 ins. = 5·58 ft. = ·085 chain. For the same reason the radius of the centre line of the four-foot way of the outer track will be greater by ·085 chain.

Example.—Suppose the lengths of leads and angles of crossings are required for a double line junction of 20 chains radius off a straight road, measuring to outsides of rails, find the particulars from the figure.

The radius of the centre line of the outer road will be 20 chs. + ·085 ch. = 20·085 chains, and the radius of the centre line of the inner road will be 20 chs. − ·085 ch. = 19·915 chains.

Find these radii at the foot of the figure, where 18-foot points are seen to be the most suitable length, and then by tracing vertically through the lines on the graph, the following particulars will be obtained from the intersections :—

	Angle.	Lead.
Crossing A and A_1	1 in 11′ − 0″	106′ − 9″
„ B	1 „ 10′ − 6″	115′ − 6″
„ C	1 „ 8′ − 0″	161′ − 6″
„ D	1 „ 6′ − 6″	197′ − 0″

To use the figure for cases where both the Main and Branch Lines are curved, it is necessary to make one or two adjustments according as to whether the curves are both in the same or opposite directions. The following equations give the *equivalent radius off the straight* for any combination of curves :—

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Let R = radius of Main Line curve
and r = „ Branch „

Then in the case of curves of similar flexure, the equivalent radius assuming a straight Main Line is $= \frac{Rr}{R - r}$

but where curves of contrary flexure occur, the equivalent radius assuming a straight Main Line $= \frac{R\,r}{R + r}$

Example.—If the centre line of the main track of a railway has a curve of 60 chains radius in which it is proposed to form a double junction to a Branch Line having a radius of 10 chains curving in the same direction, or curves of similar flexure, find from the figure the necessary particulars as to leads and crossings.

$$\text{Equivalent radius off straight} = \frac{Rr}{R - r} = \frac{60 \times 10}{60 - 10} = \frac{600}{50} = 12 \text{ chains.}$$

The figure is now referred to, and the leads and crossings corresponding to a 12-chain curve obtained. These will be found to be near approximations to the calculated results.

For very accurate work, there is, of course, nothing better than a detailed drawing.

Example.—What is the flattest curve for a Branch Line junction with a Main Line, whose centre line has a radius of 80 chains, curves of opposite flexure, the diamond crossings not to be flatter than 1 in 8 so as to conform with the Board of Trade Regulations?

The actual radius of the Main Line curve, where the diamond crossing will be fixed = 80·085 chains.

From the figure it will be seen that a 1 in 8 diamond crossing gives 21·53 chains as the flattest radius of Branch Line curve and the equivalent radius off the straight $= \frac{Rr}{R + r}$

$$\therefore 21{\cdot}53 \text{ chains} = \frac{80{\cdot}085 \times r}{80{\cdot}085 + r}$$

$$\text{or} \qquad (80{\cdot}085 - 21{\cdot}53)r = 80{\cdot}085 \times 21{\cdot}53$$

$$\therefore r = \frac{80{\cdot}085 \times 21{\cdot}53}{58{\cdot}555} = 29{\cdot}445 \text{ chains}$$

Now "r" being the radius of the road curve upon which the diamond crossing is situated, it is necessary to deduct ·085 chain in order to obtain the radius of the centre line of the Branch Line, *i.e.* 29·445 − ·08 = 29·36 chains.

Although the radius of the Branch Line must therefore be 29·36 chains it is necessary for the purpose of using the figure to assume a radius of

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21·53 chains off a straight Main Line, because the actual curves are of contrary flexure.

Any number of examples might have been given, but if the application of the principles already stated is thoroughly understood, a great saving of time will result from using the figure for connections to groups of sidings, cross-over roads, through roads, etc.

The figure is divided in such a manner as to show, at a glance, the most suitable length of switch for the various radii of curvature and crossings. As 9-foot switches are not now used in Main Lines, they are shown to be absorbed by the 12-foot switch.

The true radius of curve for a given length of switch is that of a curve which can be drawn in the obtuse angle between the switch and the Main Line, so as to be tangential to the switch at its heel and also the Main Line.

The length of switch to be used is such that the radius calculated in this manner should not be less than the radius of the curve of the junction, unless practical reasons require it to be otherwise.

Most railway companies use the following radii in conjunction with the switches in general use :—

9-foot	switches	=	430 feet	or 6·5	chains radius
12	,,	=	770	,, 11·7	,,
15	,,	=	1200	,, 18·18	,,
18	,,	=	1730	,, 26·2	,,
20	,,	=	2133	,, 32·32	,,
24	,,	=	3072	,, 46·54	,,

The lengths of switches shown on Fig. 110 are in accordance with this table and are the shortest that should be used. Occasionally it is advisable to use a longer switch, especially where the curves of a junction are in the same direction; then the next longer should be used, and the length of lead increased to suit that switch. This may be obtained approximately by adding 6 feet more or less to the length of the lead, but at the same time making sure that it is within the limits for the use of the particular length of switch. If greater accuracy be desired, the lead should be calculated, or drawn out to a scale of at least 8 feet to an inch.

Formulæ.—The following formulæ should also be considered in conjunction with Fig. 110. The notation is the same, and all dimensions are in feet (see Fig. 111).

R = radius of curve.
t = throw or clearance at heel of switch.
s = length of switch.
C = length of half-chord from tangent point to heel of switch.
V = versine to ditto.

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G = gauge or distance between outsides of rails.
D = distance between imaginary parallel line and rail on which the crossing lies.
L = length from tangent point to crossing.
M = „ „ heel of switch „
α = angle of crossing in degrees.
β = angle formed between switch and rail in degrees.

To find the length of lead and angle of crossing for a junction of given radius, and fixed length of switch, proceed in the following order :—

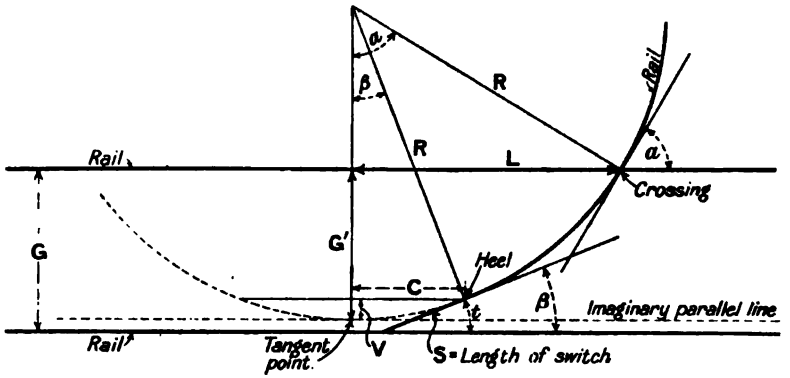

FIG. 111.

Find the point C at which the curve of the junction becomes tangent to an imaginary line drawn parallel to the Main Line rail (shown by dotted lines on Fig. 111).

$$\frac{C}{R} = \frac{t}{s}$$

$$\therefore C = \frac{Rt}{s} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (i)$$

The distance between the imaginary line parallel to the Main Line and the Main Line rail upon which the crossing lies = G'.

Now $G' = G - l + V$

and $V = \frac{C^2}{2R} = \frac{R}{2}\left(\frac{t}{s}\right)^2$ from previous figures

$$\therefore G' = G - t + \frac{R}{2}\left(\frac{t}{s}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (ii)$$

Next find the distance between the tangent point and the crossing = L.

$$L^2 = G'(2R - G')$$

$$\therefore L = \sqrt{G'(2R - G')} \quad . \quad . \quad . \quad . \quad . \quad (iii)$$

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Length of lead = L − C − s, and by substituting values obtained in (i), (ii) and (iii)

$$\text{Length of lead} = \left\{G - t + \frac{R}{2}\left(\frac{t}{s}\right)^2\right\}\left\{2R - G + t - \frac{R}{2}\left(\frac{t}{s}\right)^2\right\} - R\frac{t}{s} + s \quad . \quad . \quad . \quad \text{(iv)}$$

The angle of crossing = α and $\sin \alpha = \frac{L}{R}$

or practically $$1 \text{ in } \frac{2R - G'}{2L} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(v)}$$

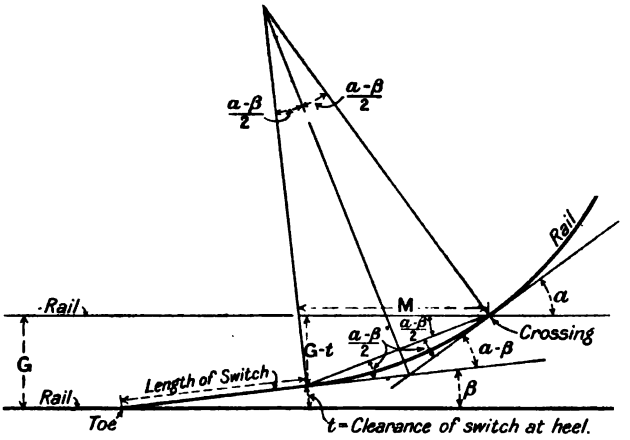

FIG. 112.

Assuming a fixed length of switch, find the length of the lead and radius of curve for a given angle of crossing (Fig. 112).

$$\text{Lead} = M + S$$

$$M = (G - t)\cot\left(\frac{\alpha + \beta}{2}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad \text{(i)}$$

$$\text{Radius of curve} = R = \left(\frac{G - t}{2}\right)\left(\operatorname{cosec}\frac{\alpha + \beta}{2}\operatorname{cosec}\frac{\alpha - \beta}{2}\right) \quad . \quad \text{(ii)}$$

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TABLE XV.

ANGLES OF CROSSINGS.

	′ ″	° ′ ″
One in	1 0 =	60 0 0
,,	1 3 =	47 9 23
,,	1 6 =	38 56 33
,,	1 9 =	33 12 11
,,	2 0 =	28 57 18
,,	2 3 =	25 40 45
,,	2 6 =	23 4 26
,,	2 9 =	20 57 5
,,	3 0 =	19 11 16
,,	3 6 =	16 25 35
,,	4 0 =	14 21 42
,,	4 6 =	12 45 32
,,	5 0 =	11 28 42
,,	5 6 =	10 25 54
,,	6 0 =	9 33 38
,,	6 6 =	8 49 24
,,	7 0 =	8 11 32
,,	7 6 =	7 38 42
,,	8 0 =	7 10 0
,,	8 6 =	6 44 40
,,	9 0 =	6 22 10
,,	9 6 =	6 2 2
,,	10 0 =	5 43 55
,,	10 6 =	5 27 32
,,	11 0 =	5 12 38
,,	11 6 =	4 59 18
,,	12 0 =	4 46 34
,,	12 6 =	4 35 6
,,	13 0 =	4 24 30
,,	13 6 =	4 14 42
,,	14 0 =	4 5 36
,,	14 6 =	3 57 8
,,	15 0 =	3 49 13
,,	15 6 =	3 41 50
,,	16 0 =	3 34 55
,,	16 6 =	3 28 23
,,	17 0 =	3 22 15

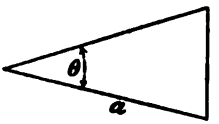

FIG. 113.

Formula

$$\frac{1}{2a} = \text{sine } \tfrac{1}{2}\theta.$$

TABLE XVI.

ANGLE MADE BETWEEN SWITCH AND SIDE RAIL.

Length of points.	Angles in degrees. ° ′ ″
9	2 23 17
12	1 47 26·8
15	1 25 57·2
18	1 11 37·4
20	1 4 27·7
21	1 1 23·5

PERIODICAL INSPECTION AND ADJUSTMENT OF THE TRACK

A railway, although originally set out most carefully by the resident engineer, is all too frequently left in the hands of the practical men, the platelayers, to maintain; and it is really surprising after a few years of wear and tear, and the so-called adjustments, to what extent the curves become irregular. Sleepers have to be packed, joints lifted, superelevation allowed for, and the line and curvature of the rails adjusted. It is, therefore, astonishing that the platelayer obtains such a good average result when it is realized that practically the whole of such work of maintenance is done with the "eye" only.

It is most important that the engineer and his professional staff should be in personal touch with every yard of curved road under his control. It is also advisable to make arrangements for the Chief

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Inspector to travel on the footplate of locomotives hauling express trains to enable him the more accurately to "feel" the condition of the track and report accordingly.

Much more than this is necessary; for the office should receive particulars, periodically, of all curves on the more important lines, somewhat in the form suggested on p. 241. This information should be obtained on the ground by the District Inspector, or somebody specially deputed to obtain it.

In carrying out this work, versines are taken with any convenient length of chord. The 66-foot length has been found very convenient in most cases, and the work is carried out in the following manner:—

Versines and Cants.—When checking the radius of a curve by means of the chord and versine, the inside edge of the higher rail should be chosen as the platelayers work to this, letting any inequality in the gauge of chairing go to the lower rail. Again, if the lower rail is chosen, in addition to the defects just mentioned, a "burr" on the outer edge of the rail is very likely to be met with, as it is on the opposite side of the rail to the flange of the wheel.

In arranging the chords each one should partially overlap the one before it, *i.e.* the second chord should be taken onwards from a point

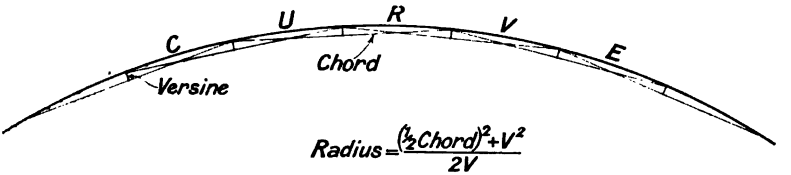

FIG. 114.

on the curve opposite to the centre of the preceding chord as shown on Fig. 114.

A chalk mark should be made on the rail where each versine is taken so that the superelevation of the rails may be obtained at each of these points subsequently.

The relative positions of all mile posts, buildings, platforms, signals, signal boxes, under and overbridges, etc., should be noted in the book, as the work proceeds, so that the office staff may take such features into consideration when studying the question of improving the track by means of sluing.

The following is a formula for finding the radius of a curve when the chord and versine are known:—

$$R = \frac{(\frac{1}{2}\text{ chord})^2 + V^2}{2V}$$

where R = radius in feet, and V = versine in feet, and the chord is also in feet.

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Table XXV., Appendix II., p. 298, has been compiled to give the radii of curves from 6 chains to 1650 chains, corresponding to offsets for a 66-foot chord. This table throughout is not strictly correct for the fractions between the inches, as the radii were computed by the formula for the whole inches, and for the fractions between these were obtained by interpolation. This is not strictly correct, but the table will be found quite near enough for all practical purposes.

Most railway engineers will find it useful to carry a copy of this and similar useful tables which have been specially compiled for this book.

Adjustment of Curves.—The form and details of notes given on p. 241 should now be examined. Assume for the moment that the last 6 columns are blank. By looking down the column giving the radii of the curves it can be observed where the curve proper commences and ends. These points should be marked in pencil and the average radius for the portion between these points computed and inserted in the column provided for the purpose. The transitional portion or run-out of the curve at each end must not be considered as forming part of the curve proper, although it is necessary, of course, for easy running and also for the purpose of gradually reducing the cant. This is usually done by $\frac{1}{2}$ inch to $\frac{3}{4}$ inch every 30 feet as stated before.

When a series of curves have been dealt with in this manner, it is advisable to check the results by means of the small scale surveys, showing the original setting out of the line; or, failing that, to compare the radii with these found on the $\frac{1}{2500}$ ordnance maps. It will generally be found that the new results compare sufficiently closely with these for all practical purposes.

The versines when tabulated in this manner often disclose the fact that a long curve in the same direction consists of some half-dozen curves of different radii. These, by simple adjustment in the manner suggested, can be converted into one with say three curves instead of six and thus produce a better running road.

Where the adjustments are only slight there is no reason why the inspector should not be entrusted with the work, since the instructions can be perfectly clear, simply stating which parts of the curve require sharpening or flattening.

On the other hand, in such cases where it is obvious that sluing rather than adjustment will be needed, in order to effect the desired improvement, there is no doubt as to the necessity of adopting other measures. In this connection it is as well to study the methods of spiralling curves, details of which are given in Chapters VIII. and IX.

When the track is on a gradient, the average maximum speed for the purposes of superelevation will be increased or diminished according as to whether the travelling is up or down. Due allowance must be made for this in the last two columns.

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NAME OF LINE..... MEASURED BY H.J.C. 2. 6. 15.

CURVE BETWEEN.. M. YDS. ..... AND .. M. YDS. .....

Mileage. M.	Mileage. Yds.	Offset to 66 ft. chord in ins.	Cant in ins. Up.	Cant in ins. Down.	Gradient in direction of mileage.	State positions of stations, platforms, bridges, points, and crossings, etc., etc.	Present radius in chains.	Adjustments required by the engineer. Offset.	Radius.	Cant. Up.	Cant. Down.	Speed in miles per hour. Up.	Speed in miles per hour. Down.
							ch:	ins.	chs.	ins.	ins.		
15	1072	$\frac{3}{4}$	$\frac{1}{2}$	1	1 in 860 falling		133			$1\frac{3}{4}$	$2\frac{1}{4}$		
	1083	$1\frac{1}{2}$	1	2			67			$2\frac{1}{4}$	$2\frac{3}{4}$		
	1094	$2\frac{1}{4}$	$1\frac{1}{2}$	$2\frac{1}{2}$			44			$2\frac{3}{4}$	$3\frac{1}{4}$		
	1105	$2\frac{1}{2}$	2	$3\frac{1}{8}$		Under bridge, No. 25	40			$3\frac{1}{4}$	$3\frac{3}{4}$		
	1116	$2\frac{15}{16}$	$2\frac{1}{2}$	$3\frac{1}{8}$			34						
	1127	4	3	$3\frac{3}{4}$		Bad length	25						
	1138	$4\frac{3}{8}$	$3\frac{1}{4}$	,,			28						
	1149	$3\frac{3}{8}$	,,	,,			29						
	1160	$3\frac{1}{16}$	,,	,,			32						
	1171	$2\frac{3}{8}$	,,	,,			42						
	1182	$2\frac{3}{8}$	,,	,,			42						
	1193	$2\frac{7}{16}$	,,	,,			41						
	1204	$2\frac{5}{16}$	,,	,,			43						
	1215	$2\frac{3}{8}$	,,	,,			35						
	1226	$2\frac{7}{8}$	,,	,,			35						
	1237	3	,,	,,	1 in 190 falling	1 IN 350 1 IN 190	33						
	1248	$2\frac{7}{8}$	,,	,,			35						
	1259	$3\frac{3}{16}$	,,	,,			31	$2\frac{7}{8}$	34	$3\frac{1}{2}$	$4\frac{1}{4}$	45	50
	1270	$3\frac{3}{8}$	,,	,,			29						
	1281	$3\frac{7}{16}$	,,	,,			29						
	1292	$3\frac{1}{4}$	,,	,,			31						
	1303	$2\frac{5}{8}$	,,	,,			38						
	1314	$2\frac{1}{2}$	,,	,,			40						
	1323	$2\frac{7}{8}$	,,	,,		3 yds past 15½ m.p.	35						
	1334	$2\frac{3}{8}$	$3\frac{1}{4}$	$3\frac{1}{2}$			38						
	1345	$2\frac{3}{16}$	3	3			45						
	1356	$2\frac{1}{4}$	,,	$2\frac{1}{4}$			47						
	1367	$1\frac{1}{2}$	$2\frac{3}{4}$	$2\frac{1}{2}$		Over bridge, No. 26	67						
	1378	$2\frac{1}{4}$	$3\frac{1}{4}$	3			44						
	1389	2	$3\frac{3}{4}$	$3\frac{1}{4}$			50						
	1400	$2\frac{1}{4}$	,,	$3\frac{3}{4}$			40						
	1411	$3\frac{3}{8}$	,,	$3\frac{1}{4}$			26						
	1422	$4\frac{1}{8}$	,,	$3\frac{3}{4}$		Bad length	24						
	1433	4	,,	,,			25	$2\frac{1}{4}$	44	$2\frac{3}{4}$	$3\frac{1}{4}$	45	50
	1444	$3\frac{3}{16}$	,,	,,			31						
	1455	$2\frac{3}{8}$	,,	,,			42						
	1466	$2\frac{3}{16}$	,,	,,			45						
	1477	$1\frac{3}{4}$	,,	,,			57						
	1488	2	,,	,,			50						
	1499	$1\frac{7}{8}$	3	,,			53						
	1510	$2\frac{1}{2}$	$2\frac{1}{2}$	3			40				3		
	1521	$2\frac{7}{8}$	2	$2\frac{1}{2}$			35			$2\frac{1}{2}$	$2\frac{1}{4}$		
	1532	$2\frac{3}{16}$	$1\frac{1}{2}$	2			45			$1\frac{1}{2}$	$1\frac{1}{2}$		
	1543	$1\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{4}$			67			$\frac{3}{4}$	$\frac{3}{4}$		
	1554	$\frac{5}{8}$	Level	Level		Reverse opp. sig. box, No. 109	159			Level owing to reverse.			
	1565	Straight	,,	,,									
	1576	,,	,,	,,									

Average radius = 34 chains

Average radius = 44 chains

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The advisability of commencing to reduce gradually the cant at the tangent point through the transition curve and onwards into the straight is dealt with on p. 123, and a practical application of this is shown in the two columns of the statement on the right-hand side headed "Cant" on p. 241.

A country gang of platelayers on a first-class English railway consists of a ganger and three men. Such a gang is responsible for the maintenance of 4 to 4½ miles of track, measured as single line, exclusive of sidings.

Where a ganger's length includes a fairly large station an extra man is provided, thus supplying about one man for every mile of running line, reduced to single line.

Each gang, of course, is responsible for the repairing of fences abutting on the length, also trimming hedges, cleaning out drains and side grips, cutting grass on slopes and so forth.

Weighing Rails in the Track.—Periodical inspections of the track are made for the purpose, among other things, of obtaining data, from which estimates and diagrams may be prepared several years in advance. These show, at a glance, the re-laying, re-sleepering and re-railing which must be anticipated.

In an important system exact weights of rails in the particular length of road under consideration are obtained by taking a typical rail out of the track, between the passing of trains, and weighing it with a portable weighing machine. As the number of years the rails have been in the road is known and also the weight per yard when new, the annual loss due to wear and tear and rusting is readily obtained.

When a rail is reduced to a particular weight in a first-class line it should be taken out and re-used in a branch line or siding. Consequently it becomes a matter of simple proportion, tempered somewhat by experience, to determine the life of the rails, and prepare a statement accordingly.

There are occasions, however, when only the approximate weight of rail per linear yard is required; in such circumstances a device, similar to that shown in Fig. 115, is very convenient. The figures representing "lbs. per linear yard" are merely for the purpose of explanation, and must not be taken as applying to any particular section of rail. It is quite obvious that a separate gauge would be required for each different section of rail in use in the railway. These figures may be obtained by calculation, or from the records in the office, but perhaps a modification of both of the results may be nearest the truth for estimating purposes, as so often occurs in practice. The lines of the device are so simple that no further explanation is needed, though in making such a device it is as well to avoid having the vertical arm too narrow, as this might cause the adjustable sliding arm to "wobble," and affect the

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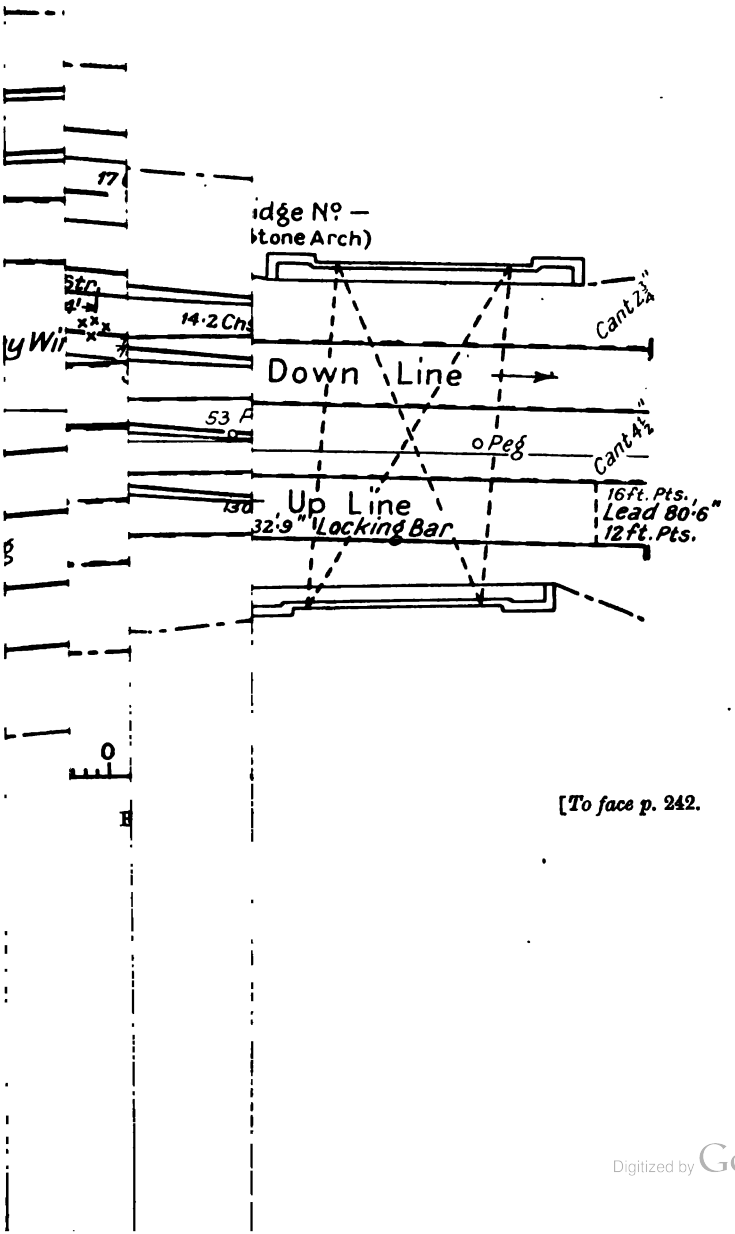

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reading. The adjustable sliding arm might be fitted with a small spirit-level, as a further precaution, provided this improvement is only used when the rail is vertical.

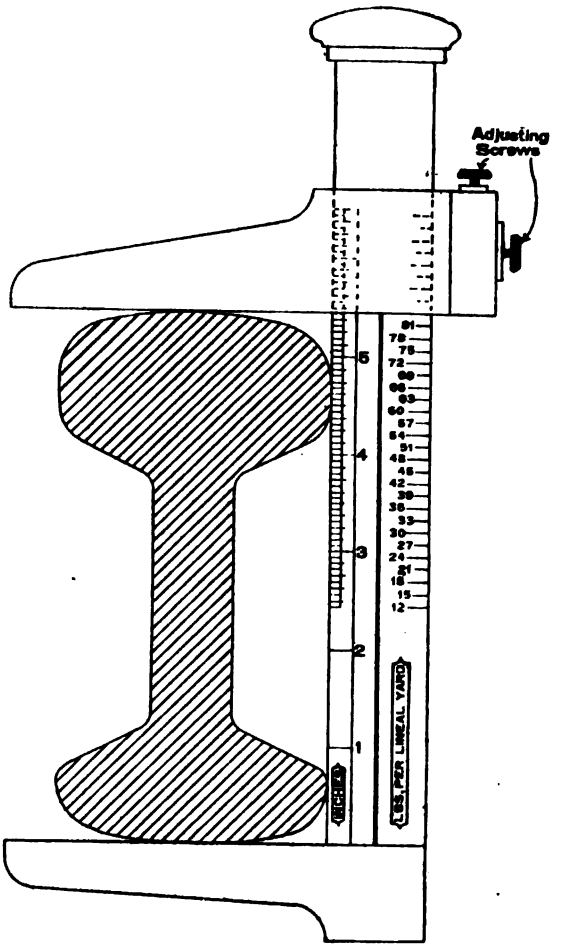

FIG. 115.—Gauge for obtaining the approximate weight per yard of rails.

SURVEYING PERMANENT WAY

An example is given in Fig. 116 of one method of scheming the improvement of an important junction. But before the position of the new permanent way can be decided upon it is necessary to make a careful survey of the line, so that a plan may be plotted to a scale of,

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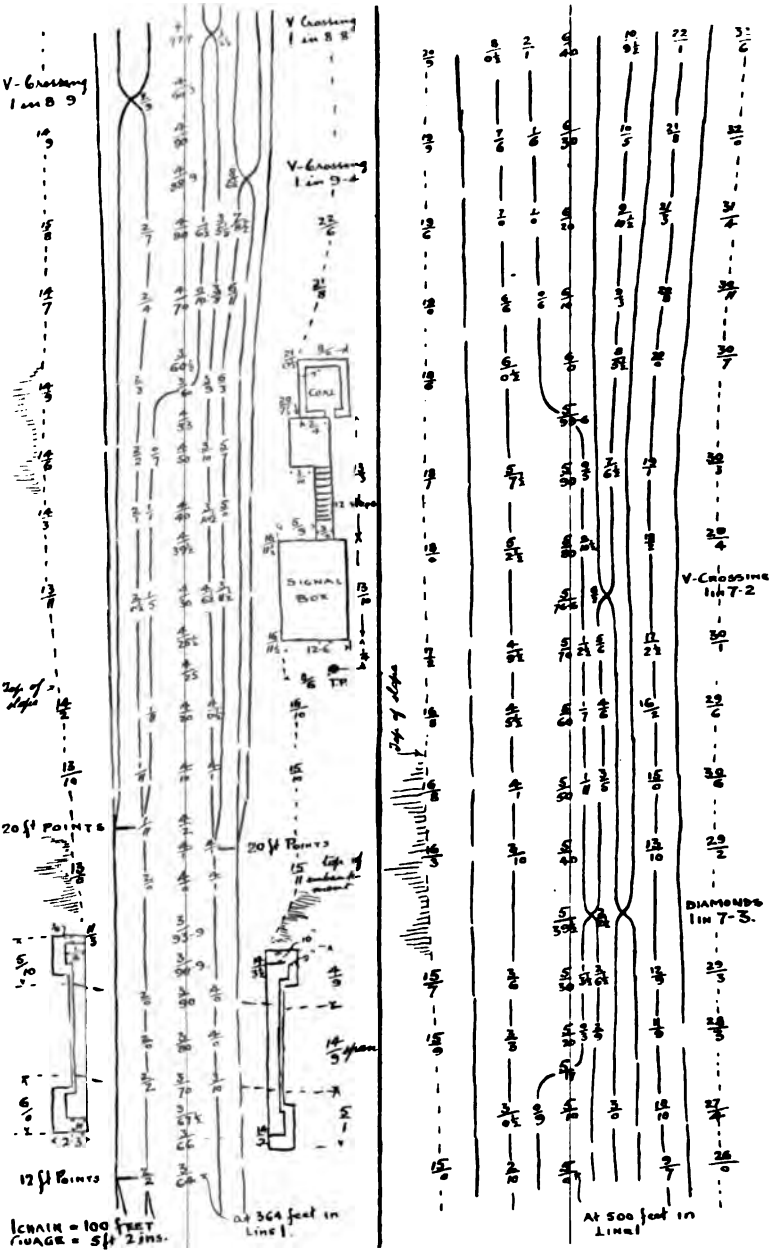

FIG. 117.

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say, 8 feet to an inch. To many experienced in land surveying this will not prove difficult, provided common sense is exercised, combined with methodical booking. Yet a detailed survey of permanent way, where the lines are joined and intersected by points and crossings, is not simple, by any means, and it is advisable before making the attempt, to plot a portion of the survey from the facsimile of the "Field Notes" given in Fig. 117. These field notes will probably seem strange to the uninitiated. One reason for this is that in practice it is found impossible to sketch the rails to their proper curves, and parallel to one another, thus making it necessary to rely solely upon the dimensions taken, and the positions of all crossings which are clearly shown. But if the particulars are plotted to a scale of 8 feet to an inch, and afterwards pencilled in, using the proper "curves sweeps" the surveyor will experience a delight second only to that of superintending the laying of a junction to his own pegs.

That portion of the field book including about 240 feet of the most complicated part of this particular survey, extending from the right-hand side of the bridge to a point well beyond the last crossing in the junction, has been reproduced (see Fig. 116).

Now with regard to the actual work on the ground it is almost impossible to make such a survey of railway connections, which is to be plotted to a large scale, without the aid of a theodolite to "set out" the straight lines ; and in this class of work pegs should be inserted in any convenient positions fixing these straight base lines. In the tops of these pegs nails must be driven after being lined-in accurately by the instrument (Fig. 118).

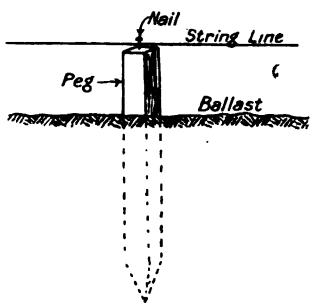

FIG. 118.

It is necessary to allow the tops of the nails to project a little above the pegs to make it possible to attach a string line to them, stretching it tightly from one to another. In this manner a line is provided on the ground from which offsets can be taken. It is necessary to see that the pegs are sufficiently high above the ground to enable the string line to pass just above the surface of the rails. By adopting this method, if the line is broken by a passing train, it is a simple matter to re-establish it. The pegs may be placed at any distances apart, within reason, so long as the nails are accurately in line.

The chain is extended in the usual way on the ballast alongside the string line, experience showing, in this class of work, that the 100-foot chain is the most convenient. In any case the length of the chain

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used must always be stated at the beginning of every fresh survey, and also the gauge of the road to which the offsets are taken.[1]

The linear dimension at about the middle of the bridge reads $\frac{3}{80}$, which means that this point is at 3 chains 80 feet, or 380 feet, from the beginning of the line. To the left and right of this figure are offsets of 2 feet and 4 feet respectively measured from the string line, and as the fellow rail to each of these must be 5 feet 2 inches away, there is no necessity to measure them. If this is remembered a lot of time will be saved, and after a little experience the habit of measuring to certain rails and no others will become acquired throughout the survey.

For plotting purposes it is convenient to have a pair of spring bow dividers fixed at 5 feet 2 inches, so that after setting off the offset on the drawing from the field book, the fellow rail can be pricked off with these at the same operation. A little pencil circle placed round the mark will be of assistance when drawing in the permanent way with the curves, etc. A careful surveyor will always check the length of his

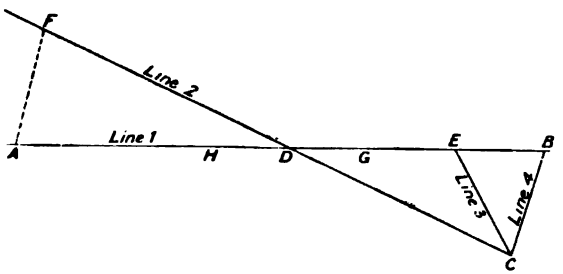

Fig. 119.

chain before commencing operations, and also afterwards, and make due allowance in the plotting for any change in length.

The accurate lengths of all chains and tapes used in surveying should be marked out with a chisel, either in the corridor of an office, on the stone flagging of a station platform, or on the plinth of a permanent building, where the ground is practically level, to be used as controls for checking the lengths of chains and tapes from time to time.

Where it is necessary to run several lines in order to incorporate the whole of a desired area, these lines should be properly triangulated, or tied together, in such a way that any error of the reading of a measurement on the ground will be discovered during the plotting of the main survey. For example, assume AB in Fig. 119 to be the main base line,

[1] A statement appears in the field notes, in this case, that 100-foot chain was used, and that the outsides of the rails were measured, giving a gauge of 5 feet 2 inches.

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and, as frequently occurs in the survey of a junction, it is found necessary to run another line in such a direction as F. For observation purposes it is advisable, in the first place, to insert ranging rods in approximately the positions indicated by the letters, in order to make sure that the course is quite clear for the string line. Pegs must be driven at the extreme ends of the lines at A, B, C, and F, and the theodolite then placed over the nail in the peg at B, and as many pegs as may be considered necessary inserted between B and A, the one at E being specially marked with a knife or an arrow stuck in the top.

Next a string line from the nail in G to that in H is run, and the instrument placed over the nail in C, and all the necessary pegs inserted between C and F. Whilst the instrument is in this position the string line, which connects G and H, will enable the peg D to be placed accurately at the intersection of the two base lines.

The survey may now be proceeded with in the manner already described, commencing at B and upon arriving at the peg E, a suitable note should be made in the field book, that at this point line No. 3 goes to the left to C. The lengths EC and BC may be measured later if desired. Upon arriving at peg D a similar note should be made in the field book, but stating that this is the intersection of lines 1 and 2.

When plotting the survey, first lay down the main base line AB and mark off the positions of E and D. Then with an ordinary or beam compass and D as centre strike an arc of a circle with radius DC. From B as centre, strike an arc with a radius BC cutting the other arc in C. Now if the measurements DC and BC are accurately taken no further dimension is really necessary, but as the value of the whole survey depends absolutely upon the accuracy of the base lines, it is always advisable to introduce a series of check lines.

In this case the line EC is a check, and if the length as scaled on the drawing is not precisely the same as that indicated in the field book, it is known at once that one of the dimensions is wrong, and a further visit must be made to the site before any time is wasted in plotting. As a further check, the dimension AF would be very valuable for the same reason as above.

If an error is made in the reading of an offset it is not so serious, as the mistake is purely local and can often be rectified in the office.

From the above plea for accuracy, the inference must be drawn that whenever it is considered important to fix any particular point accurately, such as the corner of a building, which is too far away from the survey line to judge the position at right angles, it is quite justifiable to take 3 dimensions to it. In fact, this little extra trouble is more than compensated for by the knowledge that the drawing is correct.

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Setting out Permanent Way

Advantage is taken during relaying to improve the track, including any connections which may exist; but as there is usually a great amount of work to be done, when the road is broken, in a limited amount of time, it is imperative that points, crossings, and closure rails should be accurately fitted together near the side some days beforehand, so that they may be placed in their permanent positions without delay or the possibility of alteration afterwards.

In order to eliminate error and also to save time during the actual progress of the work, the new rails should be set out by means of string lines under the supervision of an engineer.

After a little practice the connections can be set out fairly quickly, if a plentiful supply of steel pins of various lengths is at hand, together with long string lines or strong cords.

The procedure on the ground is as follows:—

From the large scale plan the position of the switches and the crossings should be fixed. A steel pin must be inserted at the heel of the switch to the standard throw of $4\frac{3}{4}$ inches from outside to outside of rails as indicated on Fig. 120, the points being straight from toe to heel.

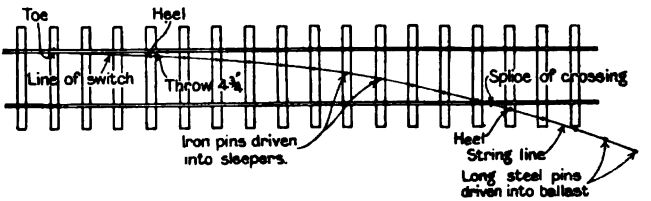

Fig. 120.

The position of the splice of the V-crossing having been chalked on the rail, the angle of this crossing denoted on the drawing should be observed and an arrow inserted at the heel, corresponding to the width shown in the standard list of crossings. Other more or less fixed points may be obtained from the drawing, and arrows should be set out at these and marked to indicate their significance. The intermediate arrows can often be lined in by eye. After a little practice it is surprising what proficiency can be obtained in this class of work.

For very precise and intricate work a theodolite should be used or else a system of offsets from chords calculated for the length of the curve, from the heel of the switch to the splice of the crossing.

Short pins are necessary for fixing in the sleepers in order to clear the passing coaches, etc., but pins from 2 to 3 feet long are required

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for placing in the ballast, especially when the road bed happens to be low.

When this work has been set out exactly in accordance with the plan, there is no reason why a slight adjustment at any particular spot should not be made, if by so doing a better road can be obtained.

In complicated work it should be considered essential to "set out" the permanent way, somewhat on these lines, to enable the foreman platelayer to take his own particulars, and obtain a thorough grasp of the details.

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CHAPTER XIII

PERMANENT WAY (*continued*)

Relaying

There are several methods of carrying out relaying, one being "to interlace the new material with the old, about 12 inches away and then slue into position as soon as the old track is removed."

Another method is to replace gradually the old material with the new, by putting the new rails into the old chairs, and afterwards taking out the old chairs and sleepers and substituting new ones, one by one.

Probably the best method, although perhaps the most expensive, is to clear away entirely all the old material, level the bed and place the new permanent way in position.

The first system is the one more generally adopted for relaying plain roads in England, as it is not nearly so troublesome as the second, and the track when completed is almost as perfect as can be obtained by the third method.

Where hundreds of miles of relaying have to be carried out every year, the most important matter to be considered is one of cost. There can be no saving of materials, and as the time occupied per yard of single line under the first system would most likely be greater than under the third, it is necessary to look elsewhere for the established economy.

The relaying of a plain road by the interlacing method can be accomplished easily between trains on ordinary working days, but to remove half a mile of track, as required by the third system, can only be attempted on a Sunday, with single line working and with from one to two hundred men working at double the rate of wages they receive on a week day. Consequently the extra cost of this method of relaying can be attributed entirely to wages.

Third System of Relaying.—It may be as well at this stage to describe as briefly as possible this method, as much of the description is applicable to the other methods. It is possible with a staff of 150 men to relay a mile of single track between 6 a.m. and 7 p.m. on a Sunday; this being the day on which possession of the line can most

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readily be obtained. But to do so, the conditions must be favourable such as would exist, say, in open country, with a fairly straight road, sufficient space alongside the track on which to lay the new material, and the presence of good existing stone ballast.

A few days before the actual relaying takes place, the ballast is removed from between the sleepers and thrown into the adjoining four-foot way. For this period the speed of all trains is slackened.

The new steel rails are laid down in the six-foot way in pairs, and marked with chalk where the new sleepers are to be placed.

The track to be relaid is the first thing broken up, all traffic being worked under single line regulations over the adjoining road. The old rails and sleepers are carried to the side of the line, and the bottom ballast loosened with picks. The ballast is then levelled ready to receive the new chaired sleepers, which will be laid approximately opposite to the chalk marks on the new rails already mentioned.

The 60-foot rails are next dropped into the chairs (about 24 platelayers are necessary to carry a 60-foot rail weighing 90 lbs. to the yard), and the fish-plates placed in position at the ends with one bolt only on each side of the joint for the time being. If everything goes on smoothly this operation should be completed by 10 a.m., and the insertion of the wooden keys follows next in turn.

At this stage a gang of six men proceed from one end to pack some of the sleepers with ballast in order to lift the road to something like its proper level. This picked gang attends to four sleepers only at equal distances apart in a rail length, as other gangs are following behind packing the remainder.

The sleepers should not be packed too much in the middle, as this is liable to cause the ends to oscillate when a train passes over it. On this account, the gang which follows, immediately, the first lifters must concentrate all their energies on packing well under the chairs, and leaving the remaining gangs to finish off.

When this work is completed throughout the whole length, it is advisable to run the Ballast Train slowly over it in order to consolidate the sleepers. Then the new ballast is emptied from the wagons and the line finally packed and straightened.

The slackening of the speed of trains should be continued for a day or two, until the road has been made perfect, as new permanent way requires frequent adjusting at first.

Expansion of Rails.—Special packing pieces are placed temporarily between the ends of the rails, before they are fished up, for the purpose of preserving a space for expansion (Fig. 121). These packings are pieces of iron bent in the form of an angle, and of varying thickness according to the temperature obtained by using a thermometer.

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Fig. 122 gives the spaces to be left between 30-foot and 60-foot rails respectively, when the temperature is known. It is based upon the assumption that the rails may be exposed in summer to a temperature of 130° Fahrenheit. The following is the formula :—

$$S = \cdot 000006 \times d \times l,$$

where S = space to be allowed in inches.

d = degrees below 130° (*i.e.* 130° minus actual temperature).

l = length of rail in inches.

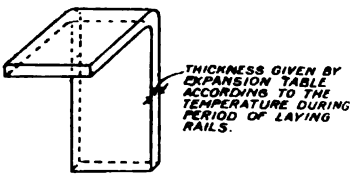

Fig. 121.

Example.—If the temperature of the rails at the time of relaying is 43°, what must be the space between the ends of the 60-foot rails before they are fished up ?

Here $130° - 43° = 87°$

$$\therefore S = \cdot 000006 \times 87 \times 720$$
$$= \cdot 375'' = \tfrac{3}{8}''$$

This amount agrees with the diagram, a copy of which should be taken on to the ground for use during relaying operations.

Future Relaying.—The general relaying, subject to slight modification, is decided upon several years in advance. The District Engineer is called upon to recommend a programme for all the relaying, re-sleepering and re-railing, which, in his opinion, is desirable on the roads under his supervision.

It is a convenience to place boards alongside the track, which state at which steelworks the rails were rolled, and the date upon which they were laid ; but in addition to this a relaying diagram should be kept in the office, and on which is recorded the renewals which have been carried out since the line was first constructed.

Every rail, of course, bears the stamp of the particular steelworks where it was rolled in addition to the date of rolling, but as it does not follow that the rails will be laid in the road in the same year in which they are manufactured, it is necessary to keep the records mentioned above.

Again, these records have the advantage that all defective rails can be traced to their source without any difficulty, consequentlya com-

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parative statement of defective rails, which have been replaced every year, together with the names of the various steelworks which supplied them, can readily be prepared for the information of those concerned.

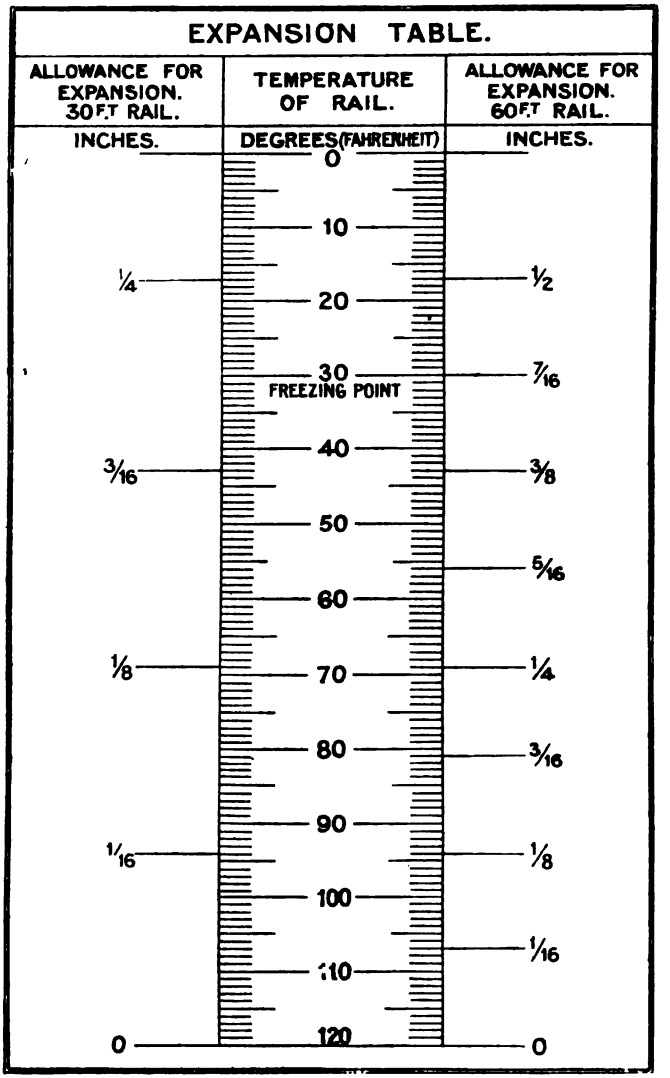

FIG. 122.

Replacement of Rails.—By the time a rail has been in an ordinary running road sufficiently long to have its weight per linear

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yard reduced from, say, 90 lbs. to about 78 lbs., it will be considered necessary to relay, probably. The weight of the rail may be found approximately by the little instrument shown in Fig. 115, or what is more desirable for precise record is to take out a rail here and there and actually weigh them with a portable weighing machine.

Where the track is subjected frequently to the action of brakes, the rails may be worn out before the sleepers; in this case re-railing only will be necessary.

On the other hand, there may be occasional cases where the sleepers first become worn and re-sleepering only needs carrying out.

In each case it is necessary to form an opinion as to the "life" of the materials which remain in the road, so that, as far as possible, all the materials will require complete renewal at the same time subsequently.

From the point of view of expenditure it is undesirable to have a large amount of relaying one year, and comparatively little the next. By careful organization it is possible to distribute the work in such a manner as to obtain a more or less constant maintenance charge, which of course is paid for out of revenue.

Material Taken Out.—A fair percentage of the old materials taken out of the Main Line during relaying are suitable for branch lines, and practically the whole of the remainder is good enough for sidings. Therefore it is economical to load up the old materials and transfer them direct either to the Branch Line or Sidings for immediate use, instead of taking them to the permanent way store yard, and causing the additional expense of unloading and stacking.

Estimating Cost.—The average cost of relaying (prior to the war in 1914) the first-class running lines in England with new materials was from 23*s.* to 25*s.* per lineal yard of single tracks after deducting a reasonable amount for credit for the old materials recovered.

It may be useful to quote a price for new permanent way alone, that is for all materials placed on top of the earth formation or road bed, including the cost of labour laying the same complete. Assuming the rock pitched formation to be 9 inches thick, good hard stone or granite top-ballast, 60-foot rails, sleepers, chairs and fastenings, the cost will work out at something near 35*s.* per lineal yard of single track.

The following is an example of the method of taking off the quantities and estimating the cost of introducing a new single cross-over road between two main running lines similar to that shown on Fig. 77. The prices quoted are always subject to the usual market fluctuations and varying cost of labour.

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ROUGH DRAFT SHOWING HOW QUANTITIES ON NEXT PAGE ARE OBTAINED.

2 pairs 12-ft. points with 30-ft. side rails
2 V-crossings 1 in 8 complete.

Screws.

	166 × 2	= 332
Points	10 × 2 × 2	= 40
Crossings	28 × 2	= 56
		428

Spikes, 428

Chairs Required.

	18 × 4	= 72
In crossings		= 6
	21 × 4	= 84
Back rail joints		= 4
		166 new chairs
		128 deduct those taken up
		38 chairs required

Felts.

	166
	38
	204

Point and Crossing Timbers.

No. 14.	9 ft. long	= 126	lin. ft.
,, 8. 10	,,	= 80	,,
,, 11. 11	,,	= 121	,,
,, 9. 12	,,	= 108	,,
,, 9. 13	,,	= 117	,,
,, 6. 14	,,	= 84	,,
,, 2. 15	,,	= 30	,,
,, 3. 20	,,	= 60	,,
		726	,,

Labour Reduced to Single Track.

North end	72 ft. × 2	= 144	lin. ft.
South end	68 ft. × 2	= 136	,,
Between crossings	10 ft. × 2	= 20	,,
In front of points	4 ft. 6 ins. × 2	= 9	,,
		309	,,
		= 103	lin. yds.

Credits.

33 chaired sleepers taken up north end
31 ,, ,, ,, south ,,
64
= 128 chairs, which will be used again.

Rails.

30′ 0″	outside rail	Down line
83′ 0″	inside ,,	,, ,,
30′ 0″	outside ,,	Up ,,
80′ 0″	inside ,,	,, ,,
223′ 0″	lin. ft.	

New rails required as closures.

No. 8.	51 ft. long	= 153	lin. ft.
No. 8.	47 ,,	= 141	,,
		294	,,
		223	,, taken up
		71	,, nett. of new rail reqd.

= 24 lin. yds., but for estimating allow 40 yds. for cutting to waste, etc.

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CHEPSTOW STATION. PROPOSED CROSS-OVER ROAD.

Plan No. 2601.

No.	Particulars.	£	s.	d.	Weight.			Amount.			Total.		
2	pairs 12-ft switches including 30-ft side rails complete	9	6	6				18	13	0			
2	V-crossings complete	8	0	0				16	0	0			
40	lin. yds. of 95-lb. rail	5	7	6	1	13	3 20	9	2	5			
726	lin. ft. of point and crossing timbers		1	3				45	7	6			
40	cast-iron chairs	3	3	0		16	0 8	2	10	7			
428	screws	21	14	6		4	3 2	5	3	9			
428	spikes	8	10	0		4	2 4	1	18	7			
214	felts per 100		6	8					14	3			
280	keys per 1000	4	1	0					18	8			
428	ferrules „	1	5	0					10	8			
44	fish-plates per ton	6	0	0		6	0 10	1	16	7			
88	bolts „	12	5	0		1	1 14		16	10	103	12	10
103	lin. yds. labour taking up main line and laying cross-over road	at	4	0							20	12	0
	Locomotive power										5	0	0
	Additional ballast										5	0	0
	Flagmen										1	0	0
											135	4	10
	Credit.												
64	sleepers taken up	1s. each									3	4	0
											132	0	10
	Add contingencies 10% plus 5% for carriage	say									17	19	2
											£150	0	0

Exclusive of Signal Department Charges.

NOTE.—These prices are based on those ruling prior to the War in 1914, and as there will be considerable fluctuations for some years it will be necessary to substitute the actual prices ruling at the time of preparing the detailed estimate, or in the case of approximation to add a percentage to the total figure.

In round figures £150 may be regarded as the probable amount of the permanent way charges in connection with providing a new cross-over road in a main running line.

It may be observed that a cross-over road is practically equivalent to two new leads, thus making one new lead cost about £75. Experience has proved this to be about correct.

A cross-over road in sidings usually cost about £100, where good second-hand materials are used, and a single lead about £50 (pre-war).

Telephonic Communications between Working Gangs and Signal Cabins.—As it is essential for the Inspector to be in constant touch with the nearest signal cabin, it is advisable to improvise tele-

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phonic communication between the parties, so that suitable times may be selected for breaking the road, sluing into position the new materials and attaching the fish-plates.

Although an experienced gang of forty men will relay, by the interlacing method, three pairs of 60-feet rails, complete with sleepers, in twenty minutes, it is not advisable to break the old road, unless the signalman is prepared to block the particular line upon which it is desired to operate for about half an hour. However, no matter how experienced the gang may be, when the traffic is resumed, it is only done so at a reduced speed, flagmen being placed at a suitable distance along the line in the direction of the traffic as a further security.

Superelevation of the Outer Rail on Curves

Fig. 123 shows the amount of superelevation to be given to the outer rail on curves from 2 to 320 chains, or from about 28° curves to 10′ curves.

In nearly all cases the cants shown by the larger figures should be adopted, but there are occasions, from time to time, when difficulties occur which render it impossible to adhere to these. At such times expert opinion is generally required before making use of the alternative cants shown in *small* figures. The ordinates of these, when intersected by the radius of the curve under consideration, give on the extreme right and left of the figure the maximum speed at which a train may traverse the curve *with safety*.

These *small* figures are based on the assumption that a train will pass round a 10-chain curve, without cant, at a speed of 20 miles an hour, but they are not intended to be used by any one except those engaged on the permanent staff.

Stress must be laid on the words "with safety" in "traversing the curve with safety," as it is not intended that these "*small* figure" cants shall be used indiscriminately, but merely as a means to an end. The following example, showing the circumstances in which they may be necessarily and usefully applied, will explain more clearly what is intended:—

The cant required on a 40-chain curve for a speed of 60 miles an hour is 5¼ inches. But if there is a physical difficulty in obtaining this amount of cant, by referring to the diagram it will be found that 5¼ inches on the "*small* figure" cants is safe for a maximum speed of 72 miles per hour. Again, the "*small* figure" cant of 4 inches is equivalent to a speed of 66 miles per hour, and as 66 is the mean between 60 and 72 would be selected most probably for the purpose.

Tables XVII. and XVIII. are compiled from Fig. 123, and will be found very convenient for pocket reference.

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TABLE XVII.

Cant of Rails on Curves.

Speed in miles per hour.	Cant in inches when radius =																		
	5 chains.	6 chains.	8 chains.	10 chains.	15 chains.	20 chains.	25 chains.	30 chains.	35 chains.	40 chains.	50 chains.	60 chains.	70 chains.	80 chains.	90 chains.	100 chains.	120 chains.	140 chains.	160 chains.
5	¼	¼	⅛	⅛	⅛														
10	1¼	1	⅜	⅝	⅜	¼	¼	¼	⅛	⅛	⅛	⅛	⅛						
15	2½	2¼	1½	1¼	⅞	⅝	½	½	⅜	⅜	¼	¼	⅛	⅛	⅛	⅛	⅛		
20	4⅜	4	3	2¼	1½	1¼	1	⅝	½	⅜	½	⅜	⅜	¼	¼	¼	⅛	⅛	⅛
25	7¼	6	4½	3⅜	2½	1⅝	1½	1¼	1	⅞	⅝	⅜	½	½	⅜	⅜	¼	¼	¼
30		8⅜	6½	5¼	3½	2½	2	1⅝	1½	1¼	1	⅞	⅝	⅝	⅝	½	½	⅜	⅜
35			9	7¼	4⅜	3½	2⅜	2½	2	1⅜	1½	1¼	1	⅞	⅝	⅝	⅜	½	½
40				9¼	6¼	4⅜	3⅜	3	2⅜	2¼	1⅜	1½	1¼	1¼	1	1	⅝	⅝	⅝
45					7⅜	6	4⅜	4	3½	3	2½	2	1⅜	1½	1¼	1¼	1	⅞	⅝
50					9½	7¼	5⅜	5	4½	3⅜	3	2½	2	1⅜	1½	1½	1¼	1	⅞
55						8⅜	7	6	5	4½	3½	3	2½	2¼	2	1⅜	1½	1¼	1
60						9⅝	8½	7	6	5¼	4¼	3½	3	2⅜	2¼	2	1⅜	1½	1¼
65							9⅜	8¼	7	6¼	5	4	3½	3	2⅜	2½	2	1⅜	1½
70								9¼	8¼	7¼	5⅜	4⅜	4	3½	3¼	2⅜	2½	2	1⅜
75									9½	8¼	6½	5¼	4⅜	4	3⅜	3¼	2⅜	2¼	2
80										8½	7½	6¼	5¼	4⅜	4¼	3⅜	3	2⅜	2¼

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TABLE XVIII.

CANT OF RAILS ON CURVES (*small* figures on diagram).

Speed in miles per hour.	Cant in inches assuming 20 m.p.h. round 10-chain curve without cant.																									
	5 chs.	6 chs.	8 chs.	10 chs.	12 chs.	15 chs.	20 chs.	25 chs.	30 chs.	35 chs.	40 chs.	45 chs.	50 chs.	55 chs.	60 chs.	65 chs.	70 chs.	75 chs.	80 chs.	85 chs.	90 chs.	95 chs.	100 chs.	110 chs.	120 chs.	130 chs.	140 chs.
15	$\frac{1}{4}$																										
20	$2\frac{1}{4}$	$1\frac{1}{2}$	$\frac{9}{16}$																								
25	5	$3\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{1}{16}$																					
30		$6\frac{3}{8}$	$4\frac{1}{4}$	$2\frac{7}{8}$	2	$1\frac{1}{8}$	$\frac{1}{4}$																				
35			$6\frac{1}{2}$	$4\frac{7}{8}$	$3\frac{5}{8}$	$2\frac{3}{8}$	$1\frac{1}{4}$	$\frac{1}{2}$																			
40				$6\frac{7}{8}$	$5\frac{3}{8}$	$3\frac{7}{8}$	$2\frac{1}{4}$	$1\frac{3}{8}$	$\frac{3}{4}$	$\frac{3}{8}$																	
45					$7\frac{3}{8}$	$5\frac{1}{2}$	$3\frac{1}{2}$	$2\frac{3}{8}$	$1\frac{5}{8}$	1	$\frac{5}{8}$	$\frac{1}{4}$															
50						$7\frac{3}{8}$	$4\frac{7}{8}$	$3\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{3}{4}$	$1\frac{1}{4}$	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{1}{8}$											
55							$6\frac{1}{2}$	$4\frac{3}{4}$	$3\frac{1}{2}$	$2\frac{3}{8}$	$2\frac{1}{16}$	$1\frac{1}{2}$	$1\frac{1}{4}$	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{3}{16}$									
60								6	$4\frac{5}{8}$	$3\frac{3}{8}$	$2\frac{7}{8}$	$2\frac{1}{4}$	$1\frac{7}{8}$	$1\frac{1}{2}$	$1\frac{1}{8}$	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{8}$						
65								$7\frac{1}{2}$	$5\frac{7}{8}$	$4\frac{5}{8}$	$3\frac{3}{4}$	$3\frac{1}{8}$	$2\frac{5}{8}$	$2\frac{1}{4}$	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{1}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{9}{16}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{8}$			
70									$7\frac{1}{4}$	$5\frac{3}{4}$	$4\frac{13}{16}$	4	$3\frac{3}{8}$	$2\frac{7}{8}$	$2\frac{1}{4}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{1}{4}$	1	$\frac{7}{8}$	$\frac{11}{16}$	$\frac{1}{2}$	$\frac{1}{4}$		
75										7	$5\frac{3}{4}$	$4\frac{7}{8}$	$4\frac{1}{4}$	$3\frac{5}{8}$	$3\frac{1}{8}$	$2\frac{3}{4}$	$2\frac{5}{16}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{1}{4}$	$1\frac{1}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{8}$	$\frac{1}{8}$	
80											7	$5\frac{7}{8}$	$5\frac{1}{8}$	$4\frac{3}{8}$	$3\frac{7}{8}$	$3\frac{3}{8}$	3	$2\frac{5}{8}$	$2\frac{1}{4}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{8}$	1	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{4}$

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Testing Speed of Trains.—It is the practice on the British railways to fix posts alongside the track a quarter of a mile apart, every fourth post indicating by cast-iron figures the full mileage from the beginning of the line or branch. This is a great convenience in stating the location of objects generally along the railway and for determining speeds is indispensable.

If the posts have been fixed accurately in position, it is possible, while travelling unobserved in the train, to record the time occupied in passing every quarter-mile post.

Where great speeds are anticipated it is advisable that two persons should be engaged, one using the stop-watch and the other booking the times occupied.

These observations should be taken once a month both on the up and down lines for at least a year, when the maximum speeds attained during that period may be tabulated. Also for the purpose of arriving at the superelevation of the outer rail on curves, it may be decided to increase the figures generally as a precautionary measure.

Again, there are speed restrictions through junctions, stations and other special places where, owing to the difficulty of obtaining the full amount of cant or to other permanent way complications, restrictions must be laid down as required by each particular case.

Unfortunately, there is abundant evidence on record, as shown by inquiries into railway accidents, that engine drivers constantly disregard these restrictions, especially when the train has been delayed owing to greasy rails, bad coal, or at stations, but such checking of speeds by stop-watch is a safeguard against the vagaries of the driver, who if reprimanded for exceeding the speed, observes the restrictions more carefully subsequently.

Table XIX., giving the speeds of trains over a distance of a quarter of a mile, has been compiled for convenience in the preparation of these records.

Superelevation of Rails in Junctions.—In a junction of the description given in Fig. 124 it is impossible to give the proper amount of cant to each road on account of the curves leading off in opposite directions. It is therefore necessary first to decide upon the relative importance of the traffic which is carried along the two lines and to assign to one of them, probably the main line, the preference.

The full amount of superelevation, in that case, will be given to the outer rails B, B, with perhaps a practical limit of 3 or 4 inches, and the permanent way engineer will decide what speed restrictions must be placed on the traffic passing to the branch line.

This can be seen to be very necessary, for the outer rail *b* of the branch line, instead of being elevated, is actually lower than the inner

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rail for that portion between X and Y in Fig. 124. Consequently in all cases of this kind the cant is on the wrong rail.

TABLE XIX.

SPEED OF TRAINS OVER A DISTANCE OF ¼ MILE.

$$\text{Formula} = \frac{\text{yards per second} \times 3600}{1760}$$

M.P.H. = miles per hour.

Time secs.	M.P.H.	Time secs.	M.P.H.	Time secs.	M.P.H.	Time secs.	M.P.H.	Time secs.	M.P.H.	Time secs.	M.P.H.	Time secs.	M.P.H.	Time secs.	M.P.H.
9	100	12	75	15	60	18	50	21	43	24	37	27	33	30	30
9⅛	98	12⅛	74	15⅛	60	18⅛	49	21⅛	42	24⅛	37	27⅛	33	31	29
9¼	96	12¼	73	15¼	58	18¼	49	21¼	42	24¼	37	27¼	33	32	28
9⅜	94	12⅜	71	15⅜	58	18⅜	48	21⅜	42	24⅜	37	27⅜	33	33	27
9½	92	12½	70	15½	57	18½	48	21½	41	24½	36	27½	32	34	26
10	90	13	69	16	56	19	47	22	41	25	36	28	32	35	25
10⅛	88	13⅛	68	16⅛	56	19⅛	47	22⅛	41	25⅛	36	28⅛	32	36	25
10¼	87	13¼	67	16¼	55	19¼	46	22¼	40	25¼	35	28¼	32	37	24
10⅜	85	13⅜	66	16⅜	54	19⅜	46	22⅜	40	25⅜	35	28⅜	31	38	23
10½	83	13½	65	16½	54	19½	45	22½	39	25½	35	28½	31	39	23
11	82	14	64	17	53	20	45	23	39	26	35	29	31	40	22
11⅛	80	14⅛	63	17⅛	52	20⅛	45	23⅛	39	26⅛	34	29⅛	31	41	22
11¼	79	14¼	63	17¼	52	20¼	44	23¼	38	26¼	34	29¼	31	42	21
11⅜	78	14⅜	62	17⅜	51	20⅜	44	23⅜	38	26⅜	34	29⅜	30	43	21
11½	76	14½	61	17½	51	20½	43	23½	38	26½	34	29½	30	44	20

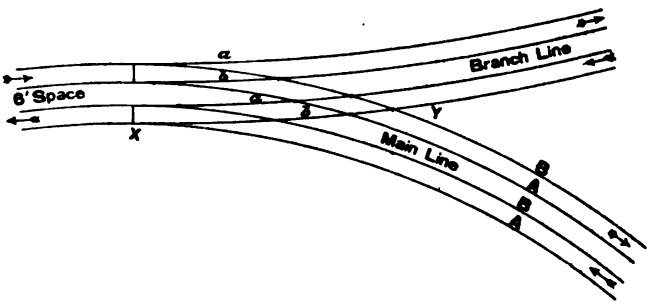

FIG. 124.

In order to illustrate this point further, let it be assumed that the main line is on an 80-chain curve and the branch line 15 chains. The outer rails of the main line will need probably 3½ inches of superelevation, and to this amount the cant of the branch line would be the wrong way. This being so, it is probable that the experienced engineer will recommend that all drivers be instructed to reduce speed to 8 or 10 miles per hour when passing through the junction to or from the branch line.

In complicated junctions, where it is desirable to provide as much

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cant as possible, rails are found assuming all sorts of different levels and much difficulty arises in timbering such places with the ordinary point and crossing timbers, because many of these timbers need to be cut or adzed in order to sink the chairs low enough, thereby reducing the strength of the timber appreciably.

It is very advisable in all cases of this kind to have exact templates prepared on the job, which will show clearly the relative heights of the different rails. From these special timbers can be prepared to suit the particular junction (see Fig. 125). The timbers shown in Fig. 125 are representative of eight special through timbers which were used in a particular place, relieving sixteen timbers of different lengths, and enabling the packing and ballasting to be accomplished more easily.

Sleepers

Since the earliest method of supporting rails on stone blocks on railways was abandoned, the wooden transverse, or cross-sleeper, has held the field. About twenty-five years ago wrought-iron and steel sleepers, rolled or pressed in the form of a trough, seriously threatened the popularity of the wooden sleeper. So many of these, after a while, were found to have cracked, and also to be unsuitable to withstand the varying conditions of the weather, etc., that they were gradually replaced by wooden sleepers, and sent back to the steelworks for remelting.

Hardwood Sleepers.—The Australian hardwoods, Karri and Jarrah, have been tried with more or less success, but it remains to be seen whether the extra cost involved will warrant their general adoption. The experience in some districts in England has been that this class of timber rots rather quickly in the middle, although the shell has the appearance of being sound. If this defect becomes general the hardwood sleeper will soon be replaced.

Ferro-Concrete Sleepers.—Again, there is the modern combination of concrete and steel, so suitable for many engineering works, which has been put forward as another competitor for use in the manufacture of sleepers. The handling of any of these sleepers, which have been tried experimentally on English railways, has been found a source of difficulty, though this would probably be overcome if they prove their worth. (See Reinforced Concrete Sleepers, p. 264.)

The favourite sleeper in England, so far, is that of Baltic red pine, and in order to ensure that each sleeper has a sufficient proportion of hardwood, not more than two sleepers should be cut from one block.

Preparation of Sleepers.—Upon arrival in this country the blocks are sawn into sleepers, and after having been stacked for about six months for seasoning, they are adzed and bored ready for creosoting.

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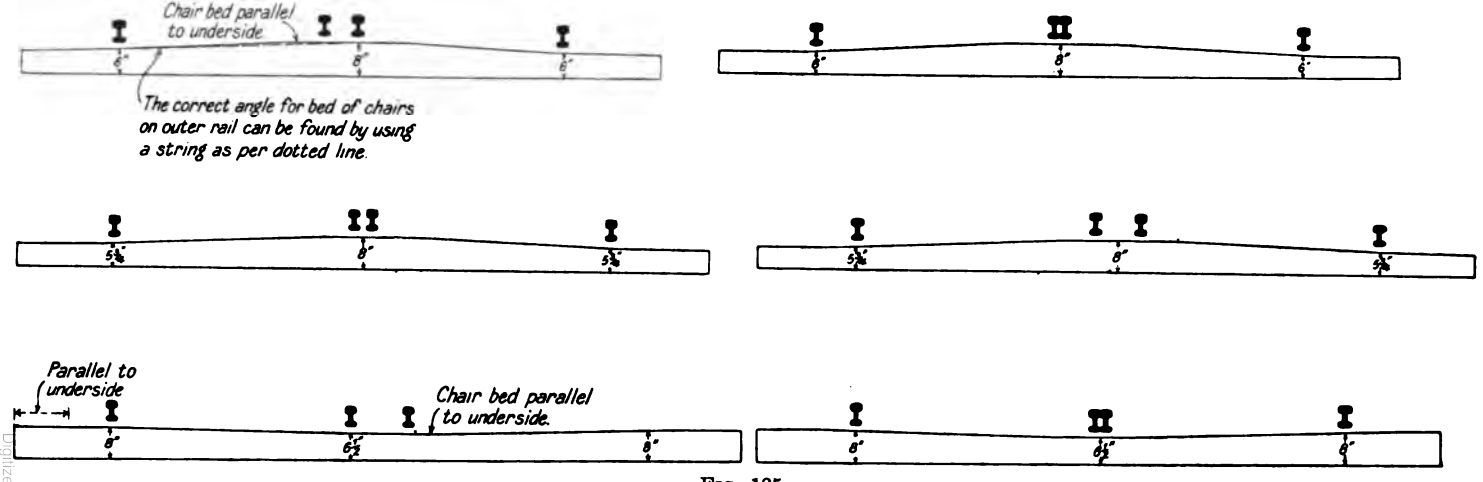
Chair bed parallel to underside.
The correct angle for bed of chairs on outer rail can be found by using a string as per dotted line.
Parallel to underside
Chair bed parallel to underside.

FIG. 125.

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The sleepers are adzed about $\frac{1}{4}$ inch deep for the cast-iron chairs, and in order to obtain a more perfect bearing, some railway companies introduce a layer of felt, about $\frac{1}{8}$ inch thick, between the chair and the sleeper, as shown in Fig. 126.

The sleeper is always placed with the heart side of its timber underneath. This is done because, if placed with the heart side uppermost, the annular rings have a tendency to open and admit water into the timber, and their dish-like direction to collect it, and so cause the timber to rot. By placing the heart portion below the water is directed away from, rather than into, the timber.

When thus prepared the sleepers are next placed on iron trucks, and run into a large steel cylinder, capable of accommodating from 100 to 400 sleepers. After the ends of the cylinder have been sealed the air is extracted to form a vacuum, and the creosote oil is allowed to enter. The oil enters under a pressure of from 100 to 200 lbs. per square inch, and when the sleepers have been under this treatment from one to two hours, about one gallon of oil per cub. foot of timber has been absorbed.

The cast-iron chairs are then put on and the sleeper is ready for immediate use.

Life of Sleepers.—Sleepers which have been properly selected and treated in this way will last on an average fifteen years in the main line, and after this period most of them are serviceable for further use in branch lines and sidings.

The dimensions of the sleepers are usually 10 inches wide by 5 inches deep, and the length 8 feet 11 inches. It is natural to anticipate the inquiry, why not 9 ft. ? The reason is, that many years ago there was a tax on all imported timber 9 feet and over in length, and the dimension 8 feet 11 inches has survived to this day.

There are other methods of preserving sleepers, some of which are patented. Among the latter it may be of interest to refer to the Haskinized process, for which a large plant was erected at Millwall Dock. In this process there is nothing put into the timber, as it is claimed that the natural juices are the right preservatives of all timber. The tree may be cut down in the morning, sawn into sizes in the afternoon, and in two days it will be ready for any work for which it is required, without any tendency to shrink or warp. The timber is put into a cylinder 110 feet long, and air pumped in at a maximum temperature of 240° Fahrenheit, and pressure of 240 lbs. per square inch. The result is stated to be similar to that from the boiling of an egg. The natural juices are claimed to be converted into fibre, and the timber appears to be hard and solid.

Reinforced Concrete Sleepers.—There are many advocates,

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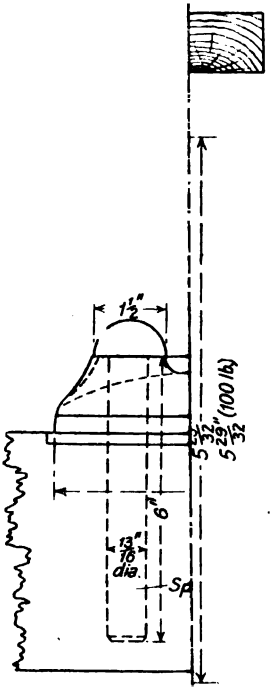

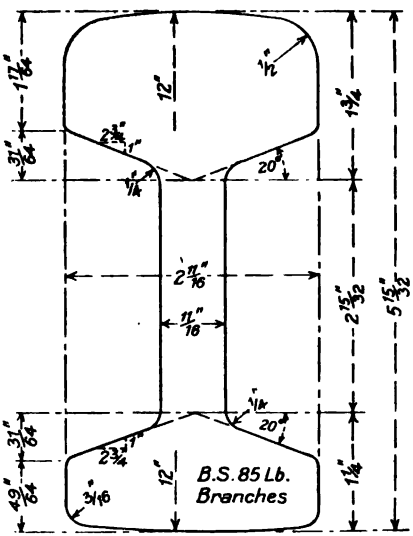

[*To face p.* 264.

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mostly designers and manufacturers, of reinforced concrete railway sleepers, but it is unlikely, at the present, that a really satisfactory sleeper has been designed, and made with this material.

Many different types of design have been laid down for experimental purposes, but some are heavy and cumbersome, others crack across the middle, and all of them seem to possess something of that ring which was noticeable when passing over steel sleepers at a fairly high speed.

In these sleepers there also seems to be a lack of resilience, or spring, which is associated with the popular wood sleeper.

Again, the timber sleepers may be roughly handled by the men without causing the slightest injury to them. Indeed, it is quite a common thing to see men, when unloading, throw sleepers, without mercy, from the top of the wagon to the ground. It is not certain that the reinforced concrete sleeper could stand that kind of treatment.

However, if the design can be made satisfactory, and the attachments properly protected from early deterioration, it might reasonably be expected that they would last much longer than wood sleepers, and so justify the extra care required in handling them. If it can be proved that the life of the reinforced concrete sleeper is double that of the wood sleeper, then the extra initial cost will show an economy, as resleepering will be saved, and other incidental expenditure.

When preparing a statement of comparative cost, it is necessary to remember that wood sleepers will do duty in second-class running lines and sidings, after being replaced in the main line, also when finally discarded for railway work they will show a credit of a few shillings each for firewood.

Spacing of Sleepers.—Like many established practices on the leading railways, the spacings of the timber cross-ties, or sleepers, has been a matter of evolution. For many years the standard distance apart, centre to centre, was 3 feet, but owing to the increased weight of locomotives, and traffic generally, together with the greater speed demanded nowadays, it became necessary to lay down heavier rails, and place the sleepers closer together.

Fig. 127 shows an example of the spacing of the sleepers on a first-class line of a leading railway company. The spacing of the sleepers closer together, near the joints of the rail, must be specially noted, as it is at such points that serious hammering takes place every time a wheel passes over.

Sleepers under Rail Joints.—There has been a great variety of devices for overcoming the annoying jolt which is noticeable in the train when the vehicle passes over the joints. None of these appear to have passed the experimental stage, and most engineers are now

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devoting their energies to making the old-fashioned butt joint as perfect as possible. This accounts for the increase in size of the sleeper on each side of the joint, heavier chairs on these particular sleepers, and an increase in the strength of the fish-plates and fastenings.

The use of welded joints is impossible on account of expansion and contraction, and the splayed or scarfed joint was found to be no improvement on the butt joint. The splayed rails caused trouble when they had to be cut to shorter lengths for fitting purposes.

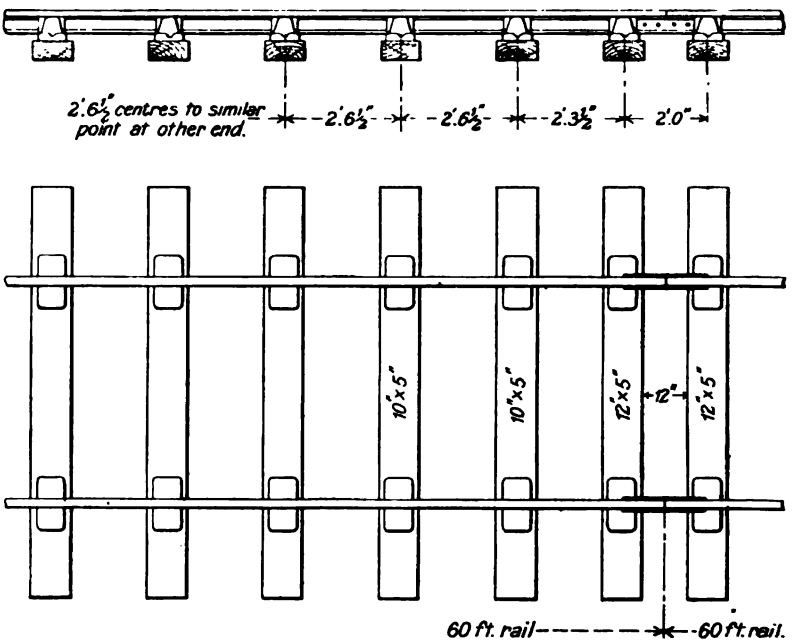

FIG. 127.—Diagram showing spacing of sleepers, first-class lines, 60 ft. rails, 24 sleepers to each length.

Again, joints have been placed in a cast-iron chair, secured to a large sleeper or timber, with complicated malleable cast-steel attachments, but the results were no better than from the comparatively simple fish-plates, as shown in Fig. 106.

On the other hand, the joint sleepers require constant attention from the platelayers by lifting and packing, and although the effect of the hammering is reduced by bringing such sleepers closer together, it is necessary to remember to leave sufficient space between them to permit of a pick and shovel being used freely.

Hammering at Rail Joints.—The rail which is called the "running-on" rail always wears rather quicker than other portions

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at the point marked X (Fig. 106), and the platelayers keep a stock of hoop iron, which they cut and bend in the shape of an angle for the purpose of filling up the small space caused by the wearing of the surfaces between the fish-plate and the underside of the top flange of the rail, as indicated at X on Fig. 106.

Fish-plate Bolts.—One end of the shank of the fish-plate bolt should be formed with a projection which is intended to engage with suitable holes in the fish-plates in order to prevent the bolts turning when the platelayer is engaged tightening the nuts with a long-handled spanner.

Chair Fastenings.—A very popular method of securing the cast-iron chairs to the wood sleepers is by means of two spikes and two screws shown in Fig. 128, but owing to the heavy traffic and increased speeds, in new tracks spikes are not used at all, three screws being found to answer the purpose much better.

The screws, however, must never be driven down to the chair, but a wooden ferrule be placed in the hole in the chair to act as a sort of cushion between the screw and the chair. This is shown in Fig. 129.

The platelayers tighten these screws from time to time, and when the ferrule has been compressed until the clearance is similar to that shown in the second illustration, it is necessary to replace the old ferrule by a new one as shown in the former illustration.

Ballast

In the early construction of railways, almost any kind of rubbish was considered good enough for ballasting purposes. And even now some very doubtful material is seen occasionally on main lines, but the probability is that the relaying and reballasting has not been completed throughout the system since the time when poor qualities of gravel were accepted.

Cinders, and other material which come within a similar category, never can be really good materials for ballasting a modern railway. They lack the strength and body which a proper "topping" should possess. Indeed, on cinder roads, not only at the ends of the rail but also in the middle the cinders soon churn up into a black mud under the sleepers, rendering the proper repairing of the road bed practically impossible.

The railway track bed of to-day must be made exceedingly strong and substantial, and only the most suitable materials are now used in the main running lines.

Foundation.—After the formation has been brought to the proper level either by excavation or filling, as the case may be, it is necessary to

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provide an adequate foundation by seeing that the substructure is perfectly secure.

Rubble pitching 10″ to 12″ thick makes quite a good foundation, and Fig. 130 illustrates the manner in which it should be laid. If the

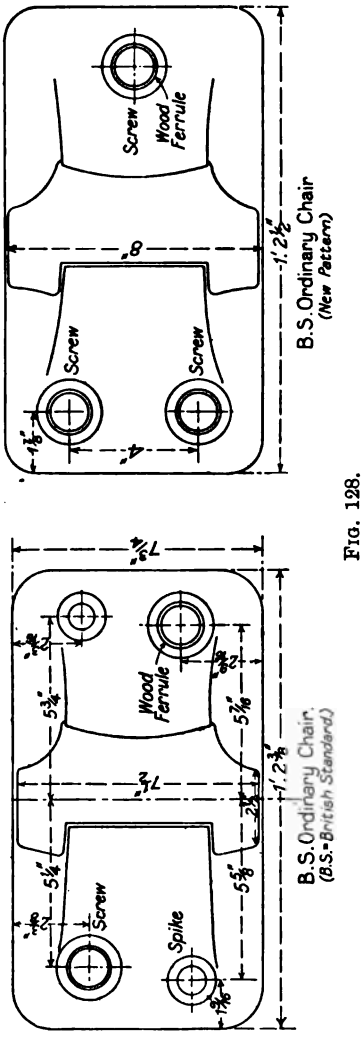

FIG. 128.

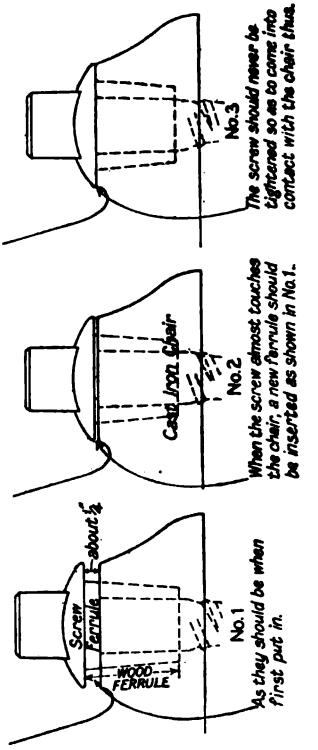
No.3
The screw should never be tightened so as to come into contact with the chair thus.
Cast Iron Chair
No.2
When the screw almost touches the chair, a new Ferrule should be inserted as shown in No.1.
Screw
Ferrule
about ½"
WOOD FERRULE
No.1
As they should be when first put in.

FIG. 129.

cost of obtaining hard rubble pitching is excessive, it may be possible to substitute some of the more durable kinds of slag from steelworks, or other material from which a suitable foundation can be formed.

Upon the pitching it is usual to spread a few inches of broken

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granite or other hard stone, and then when the sleepers are laid and the rails secured, it is usual to ballast them up to the surface as shown.

Specification.—A short specification for broken granite ballast is as follows :—

The whole of the stone shall pass through a $1\frac{1}{2}''$ diameter ring, 10 per cent. of it may pass through a $\frac{1}{2}''$ ring, and 1 per cent. of that 10 per cent. may pass through a $\frac{1}{8}''$ diameter ring. It means a small graded stone and one there is no difficulty in moving with a good shovel.

Wet Formation

A typical example of first-class railway construction is shown on Fig. 130, and the case in review was where the road was in cutting with the stone foundation or pitching laid on clay formation.

The cutting turned out to be rather wet, and eventually a 9″ pipe drain was laid along the six-foot way, with 6-inch branch pipes leading to the side grips or pipes. These drains were effective in carrying off a certain amount of the surface water, but the top clay eventually began to churn up through the pitching and into the ballast, making it extremely difficult for the platelayers to maintain a good road. As time went on the track seemed to settle, and additional ballast was placed on the top for levelling-up purposes. The old trouble of clay "oozing up" again appeared, and upon examination it was found that the original pitching had become buried completely in the clay. It was decided to place a fresh layer of pitching on top of the old, and top-up with new ballast. This was fully expected to successfully remedy the trouble, but failed to do so, and in reality proved a very expensive form of "patching." It was necessary to find a remedy and accordingly an experiment was

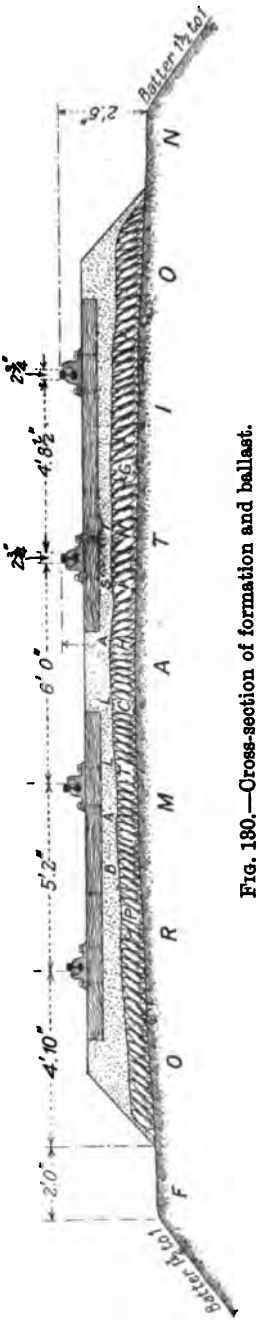

Fig. 130.—Cross-section of formation and ballast.

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tried, on a length of about a quarter of a mile, by laying a 3-inch layer of fine cinders, specially obtained from the engine sheds, evenly on top of the clay. New pitching was then placed on top of the cinder coating and the bed was finished off with ballast in the usual way.

Careful observation was kept on the road, and when a period of five years had elapsed and it was seen that the surface ballast was still clean, it was decided to examine the foundation and discover what changes had taken place. The ballast was removed for a length of about 15 feet, and the pitching when exposed was found to be quite clean. Underneath the pitching, the layer of cinders had become caked and had had the effect of hardening the surface of the clay. The remedy was considered a complete success, and the rest of the road affected by the "oozing" trouble was treated in the same manner with excellent results.

This is an instance of a very successful method of remedying a particular wet road bed, but it must not be supposed that the same remedy will apply in every case. Engineers must be resourceful in order to be successful in all their enterprises, and must first diagnose each case carefully, and then encourage their ideas fully to suggest a suitable remedy which must always be tested experimentally before being finally adopted. A good foundation is essential for the satisfactory maintenance of any road.

Subsidence

Although subsidences of railways due to mining operations are more or less anticipated, the actual settlement is generally accompanied by a little excitement and extra watchfulness, even though the movement be very slow. It is difficult to predict when it may develop into a serious matter, and even the platelayers seem to awake from their lethargy and assume an air of personal responsibility.

Coal Workings.—All the railway companies are in possession of correct plans of the coal workings under every portion of the lines which they control. These plans are prepared either by their own mining engineers or some mining engineer, in private practice, specially engaged for the purpose. The law demands that notice in writing be given to the railway company a reasonable time before operations are commenced at a particular "working." This obligatory notice is generally accompanied by more precise information as to the depth and thickness of the seam to be worked, and the plans are amended from time to time by the mining engineer.

As opportunity occurs the railway engineer arranges for large quantities of cinders to be tipped on each side of the track as a precautionary measure, and as the permanent way settles, the line is packed up from time to time as required.

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Notifications of Subsidences.—When subsidences occur all the departments concerned will be notified and arrangements made in the weekly notices for the slackening of speeds. Where necessary, detonators may be placed on the rails by the flagmen, specially to remind engine drivers of the slackening.

The weekly notices just referred to are compiled from information furnished beforehand by the different departments, which have work to do on the railway, such as permanent way, repairs, and renewals of bridges, signals, etc. Engine drivers, traffic inspectors, signalmen, and others interested are supposed to make themselves familiar with all the printed matter relating to their districts.

Slips

Slips are very closely related to subsidences except that they, not infrequently, take place without any previous warning.

In some cases an embankment may have given signs of movement long before the actual collapse takes place, whilst in others the first intimation comes from the ganger or engine driver who reports the matter to the nearest signal cabin.

Flagmen are immediately called out to either stop or slacken trains and the engineer arranges for as many trains of cinder ballast to be sent to the spot as are thought necessary to pack the road temporarily.

The cinders are unloaded along the whole length of the slip, both in the six-foot way and alongside the line, and the permanent way is packed until the desired level has been obtained, watchmen, of course, remaining on duty as long as required.

Cinders used in this way will very often remedy the evil, but when the slip is serious and likely to give further trouble, the only effectual way of dealing with it is to drain the bank thoroughly.

Counterfort Drains.—This can be attained by constructing rubble counterfort drains from 4 to 7 feet wide, and from 14 to 30 feet centres as occasion may require. The excavations in the slope should be continued down to a depth of about 3 feet below the bed of the slip, *i.e.* below the wet slurry. When this point is reached the rubble is thrown in and the shoring or trench timbers withdrawn in the usual way, at the same time putting in drain pipes to conduct the water to a suitable outfall. A section of a counterfort drain actually constructed a few years ago in a very high and important embankment carrying a main line is shown on Fig. 131.

The surface of the slope is reproduced from actual levels, and the bulge towards the bottom proves undoubtedly that the bank had been more or less unsettled from its inception about forty years previously.

The above method of constructing counterfort drains may appear

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to the uninitiated extravagant, but when there is certainty of success, the mere question of cost should not be allowed to intervene, where the risk of accident is so great. There are many things in connection with railway engineering in which economy may be introduced with advantage, but slips and subsidences do not come within that category.

As an alternative to the straight counterfort drains running down a bank, branch drains of the "Y" form on plan are sometimes adopted, and such drains can frequently be seen when travelling in trains. It does not follow that wherever these drains exist there must have been a slip necessarily, because if there is any apprehension as to the stability of the bank when in process of construction, owing to the presence of water or for other reasons, the precaution would be taken of introducing such drains at suitable intervals. For these drains rubble of irregular sizes is the most suitable, the largest pieces being no heavier

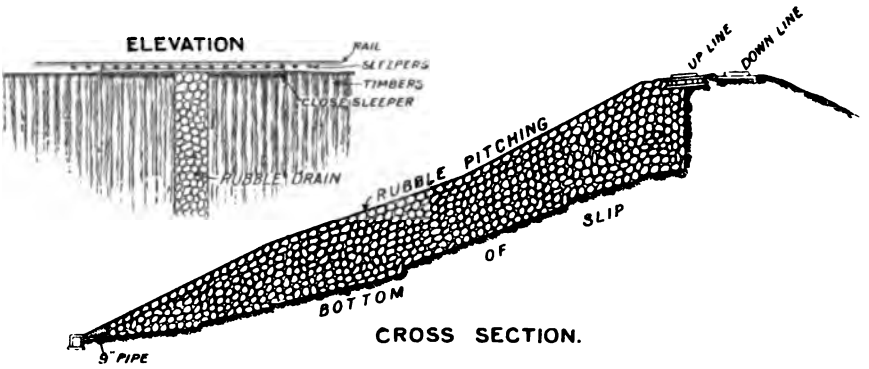

FIG. 131.

than what a man can lift and place, say, 70 lbs. Slag from ironworks or any hard and durable material will do quite well, especially if it can be obtained quickly and the price is satisfactory.

Temporary Expedients.—Before commencing the excavation for drains where a slip has occurred, it is advisable and indeed necessary, to place stout longitudinal timbers, under the existing sleepers which carry the rails, of sufficient length to be well clear of the trench, say, 30 feet to 40 feet long. Also, in order to make perfectly certain that the longitudinal timbers have a proper bearing, second-hand sleepers should be placed close together under these timbers as shown in Fig. 132.

Fig. 132 show a useful arrangement of timbers for emergency purposes, but modifications, if necessary, can be introduced easily in the details, so long as the object is well understood, namely, distribution of load, on suitable formation, for the purpose of safety.

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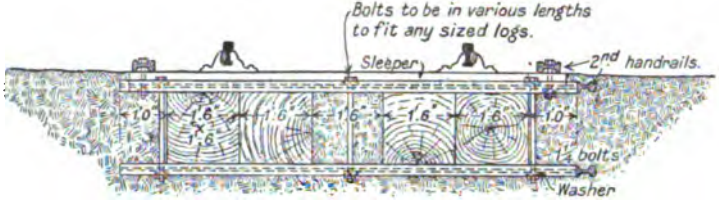
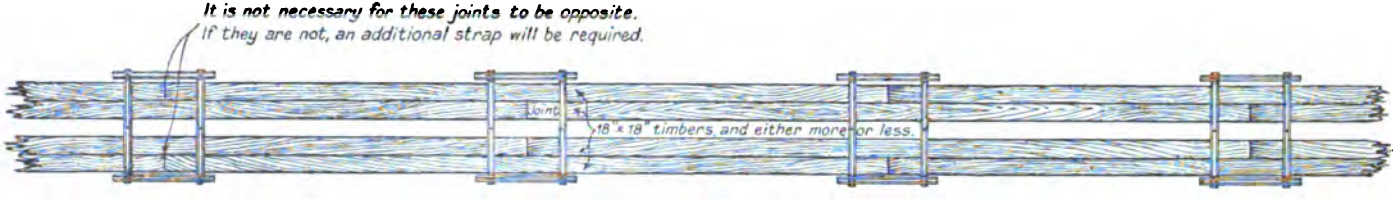


Section.
Scale: 4 feet to an inch.


It is not necessary for these joints to be opposite.
If they are not, an additional strap will be required.
Joint
18" x 18" timbers, and either more or less.

Plan
Scale, 16 feet to an inch.

FIG. 132.—Temporary timbering to be adopted for embankment slips, etc.

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Probable Causes.—From careful observations made of some recent slips, it appeared that when the embankments were formed the earth was tipped sideways, and always at nearly the full height, the consequence being that the largest pieces of clay would roll to the bottom, the finer and softer material remaining in the middle. The clay at the sides would, no doubt, prevent the escape of water, as it becomes after a time more or less impervious, and the continual vibration caused by trains passing over, would have a churning effect upon the softer material forming the inside of the embankment, and ultimately lead to movement. It is the wet stuff in the middle which must be tapped in some way.

Forming Embankments.—A very good method of forming a high embankment is to have three separate temporary roads diverging

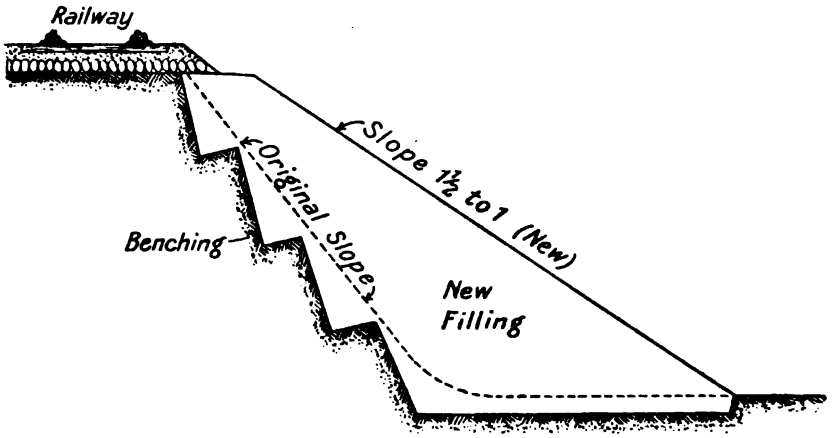

FIG. 133.

from the overland sidings and to tip endways, keeping the two outside tips the more advanced, while the middle one follows on and acts as a kind of key.

Another method, and one which is frequently adopted, is to tip in layers about 15 feet deep, taking special care that the bottom layer, and, in fact, each succeeding one, is kept to the full width of the embankment.

Slips are primarily due to water finding its way into and accumulating in the embankment, or they may be due to a poor clay which is sodden with water, sliding on the original surface of sloping ground. However, in good work it is usual to bench the surface of the original sloping ground in order to give the tipped material a better hold.

If an existing bank is being widened, it is necessary to cut benchings in the original slope to enable the new filling to unite with it (see Fig. 133).

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The height to which an embankment should be taken above what is permanently required, depends upon the material used, and also upon the weather during the period of work. Some experience is necessary to decide upon this amount, but, speaking generally, if an embankment, say, 20 feet high and 100 feet wide is formed of clay in changing weather, it should be taken about 15 inches higher than the desired permanent level to allow for settlement. If the work is done in dry weather about 24 inches should be allowed; and if the clay has been excavated by a steam navvy, it is necessary to allow even as much as 30 inches. A common rule which applies in the case of clay, pretty generally, is to allow an extra inch for every foot of height of embankment, for settlement purposes.

An embankment of sand and gravel settles very little, and it is rarely necessary to allow more than a few inches.

On all embankments a uniformly distributed thickness of soil of not less than 6 inches should be spread over the surface of the slope and sown with grass seeds in the proportion of 30 lbs. of rye grass and 15 lbs. clover to each acre.

Platform Copings

The standard height of platforms is 3 feet above the surface of the rails, and the coping is fixed at a distance of two feet from the outer edge of the nearest rail, on a straight road. Therefore, where the track is curved, an allowance must be made for the versine and cant, which will necessitate the coping being set back a certain distance, varying with the radius of the curve, and the amount of the cant.

The standard dimensions of the largest rolling stock being known, it is only necessary to calculate the normal position of the footboards when passing round a particular curve. But the effect of vibration on the springs of the vehicles must not be overlooked, and perhaps the most reliable method of setting out a new platform coping, next to using the vehicle itself, is to make a templet.

Assuming that the curve is correctly laid out, and knowing the wheel base of the largest vehicle, it is not a difficult matter to make a templet, defining the outside limits of the footboards, and after making due allowance for clearance the coping may be fixed in position.

The introduction of longer vehicles from time to time involves the setting back of many of these platform copings, which are situated on fairly sharp curves. In all these cases the templet is the most convenient method of arriving at the proper clearance.

It must not be overlooked that the older pattern vehicles, having a shorter wheel base, and a smaller width across the footboard, would show a rather wide space between the nose of the coping and the edge

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of the footboard, and be a source of danger to the public, consequently it is advisable to make a not too generous allowance for clearance.

Foundations under Water Troughs

On the main lines of the more important railways it is usual to lay water troughs in the four-foot way, which have a length of about half a mile, so that engines on the long-distance runs may pick up water whilst travelling.

It is quite to be expected, that every time the scoop is used a considerable quantity of water flies on to the permanent way, and precautions must be taken to carry away this waste water as quickly as possible, without damage to the track.

Large-sized pitching is laid as a foundation under the troughs, and for a considerable distance beyond each end, the depth being 2 feet 6 inches in the centre, and 4 feet towards the outside of the track. The overflow water percolates through the voids in the pitching, and finds its way eventually into the side drains (Fig. 134). It is found, however, that after being in constant use for four or five years there is a gradual deterioration in the filtering qualities, probably caused by the voids becoming choked with dirt.

There is plenty of room for improvement in the formation of the track in such situations, and the defect above referred to naturally leads to the question why the overflow water should be allowed to penetrate in such large quantities below the bed of the sleepers.

It is quite unlikely that an impervious road bed has ever been tried, and although there are many possible ways of constructing such a track, a carefully applied scheme of reinforced concrete suggests itself, as one which would be likely to be very successful, especially if the finishing layer of fine concrete has been waterproofed by one of the many proprietary brands on the market. With reference to this latter point, if something approaching a perfect result is to be obtained it will be necessary to use asphalt, which is, without doubt, acknowledged to be the best waterproofing material for such a purpose.

There would be no difficulty in carrying out these ideas on a new railway, but it would be a rather tedious operation to substitute reinforced concrete under the water troughs on existing lines, where traffic must proceed as usual. No doubt, if absolutely essential, even this is not insurmountable.

Concluding Remarks.—It is very desirable and, indeed, necessary that all students of railway engineering should seize every opportunity of witnessing work in progress.

The chief engineer, of course, will see personally that his pupils are instructed properly in most things that matter from a practical point of

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view, but there are many apprentices and draughtsmen whose knowledge and ability would be improved materially if their hearts are in their work, by their getting into touch with the inspectors, and arranging to accompany the staff.

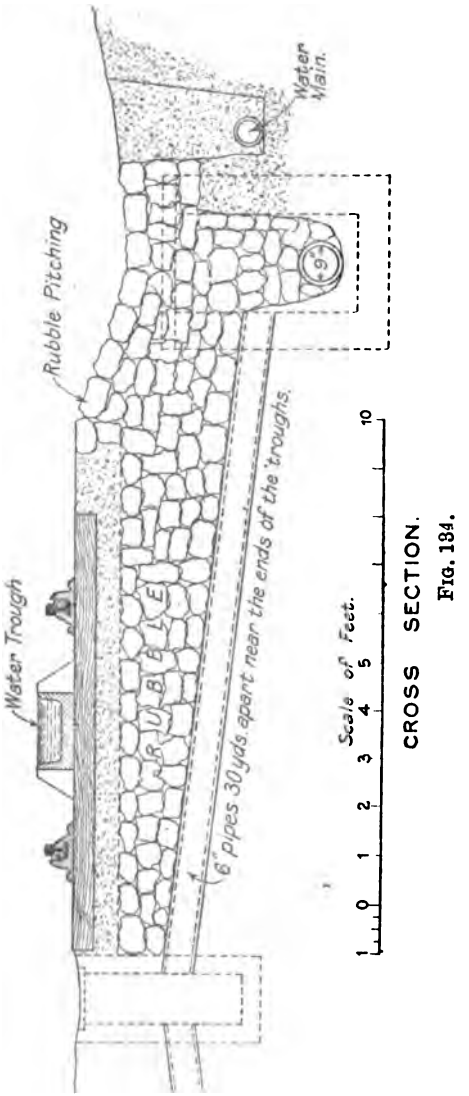

FIG. 134.

The operations in connection with the relaying of a junction, or the renewal of a bridge, are commenced usually between 4 and 6 a.m., and,

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although it may be possible to reach the point by passenger train later in the day, the educational value of the visit is considerably discounted by a late arrival.

It is strongly to be advised that the ambitious engineer start with his men, and make notes, sketches, and records of the times of the various stages of the operations; tabulating these and other impressions at the opportune time within the following few days.

The true expression of engineering work is only obtained by the proper application of theory and practice; yet how frequently one discovers in designs a serious lack of practical knowledge, which never fails to impart a feeling of doubt, in the mind of the foreman, as to the successful issue of the work.

There are endless opportunities of adding to, and improving, one's knowledge and ability, and the young engineer who "makes good" on the lines suggested is paving the way to his receiving a due reward sooner or later.

The draughtsman commences business at 9 a.m., and finished probably at 5, or 5.30 p.m., and considers himself a free lance until the following day. Such men possibly may serve a useful purpose according to their lights, but surely the "light" seems a very dim one when thinking of the awful monotony of "pushing" a Tee square to the end of one's days.

With all its ramifications civil engineering is one of the most interesting and delightful pursuits, but no man has ever risen in the profession to any position of importance and responsibility without hard work, and plenty of it.

Accordingly, it is necessary to impress upon all how essential it is to see actually as much work in progress as possible, in order that their theory may become a live force, ready to act in conjunction with their practical knowledge in the production of real, natural, and stable structures.

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APPENDIX I

Barometric Hypsometry and Tables

As aneroid barometers are graduated by comparison with a standard mercurial barometer, it will only be necessary to deal with the latter here, since all barometric tables are constructed for mercurial barometers.

If changes in elevation accounted for all the variations in the readings of a barometer, no correction after that for the initial datum would be necessary, and the instrument would be of much greater value for levelling purposes. Also a device by means of which elevations could be read to inches would have been attached to them by now. Unfortunately barometric readings indicate pressures of the atmosphere, the density of which latter is not homogeneous. Also temperature plays an important part in causing the need for corrections.

The United States Coast and Geodetic Survey have contributed the results of very exhaustive investigations of the use of the barometer for levelling purposes to the scientific world. This is set out in their 1881 report in Appendix X.

Perhaps the principal lesson learnt from these researches is that "the greatest uncertainty in barometric hypsometry arises from the imperfection of temperature corrections." Results vary very much with the seasons of the year, and also with the time of day. In the warmest season the results are too great, as also in the warmest part of the day. In a similar though contrary manner they are too small in the coldest season and part of the day.

It was discovered that these discrepancies arose from the erroneous assumption that the average temperature of the air column, upon which the difference of pressure between the two stations depends, is the average of the observed temperatures at these stations.

Table XX. is taken from Table XI. in Appendix X. of this report, and is derived as follows:—

Let A = height of atmosphere (assumed homogeneous) in the average latitude of 45°.

r = corresponding height of barometer.

r_u = reading of barometer at upper station.

r_l = reading of barometer at lower station.

h = difference in elevation between the upper and the lower station.

Now the relative density of the imaginary homogeneous atmosphere with reference to mercury $= \frac{r}{A}$.

This may be assumed constant and $= \delta$.

$$\therefore = r = \delta A$$

An infinitesimal change in h, namely dh, will bear the same ratio to dr as A does to r.

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$$\therefore dr = \delta dh$$

$$\therefore dr = \frac{r}{A} dh$$

or $$dh = A \cdot \frac{dr}{r}$$

If this be integrated between the limits r_u and r_l we have

$$h = A \cdot \int_{r_u}^{r_l} \frac{dr}{r} = A \cdot \log_e \frac{r_l}{r_u}$$

or $$h = A \cdot \log_e 10 \cdot \log_{10} \frac{r_l}{r_u}$$

$$\therefore h = A \cdot 2{\cdot}3026 \cdot \log \frac{r_l}{r_u} \quad \ldots\ldots\ldots \quad (1)$$

in the common system of logarithms.

To find a value for A it was necessary to find experimentally what A would become at a temperature of 32° F., and where gravity was taken into account. Knowing the specific gravities of mercury and air a relation could be obtained between A and the height of the barometer at sea-level at 32° F. In this way and taking account of gravity approximately A_0 (at sea-level) was found to have a value of 26,284 feet. But the coefficient of expansion of air for 1° F. = ·002034.

Therefore for any other temperature than 32° F. we have

$$A = 26284[1 + {\cdot}002034(t - 32°)];$$

or where t is the average of the temperatures at the upper and lower stations t_u and t_l, we have

$$A = 26284\left[1 + {\cdot}002034\left(\frac{t_u + t_l}{2} - 32\right)\right]$$

or $$A = 26284[1 + {\cdot}001017(t_u + t_l - 64°)]$$

Substituting in equation (1) we get

$$h = 26284[1 + {\cdot}001017(t_u + t_l - 64°)]2{\cdot}3026 \cdot \log \frac{r_l}{r_u}$$

$$\therefore h = 60521{\cdot}5[1 + {\cdot}001017(t_u + t_l - 64°)]\log \frac{t_l}{r_u}$$

Taking the barometer reading at sea-level to be 30 inches, and assuming the mean temperature of the two stations to be 50° F., this last equation may be written—

$$h = 60521{\cdot}5[1 + {\cdot}001017 \times 36°]\log \frac{\frac{30}{r_u}}{\frac{30}{r_l}}$$

$$= 60521{\cdot}5[1 + {\cdot}001017 \times 36°]\left[\log \frac{30}{r_u} - \log \frac{30}{r_l}\right] \quad \ldots \quad (2)$$

The values given in Table XX. represent one portion of this quantity, *e.g.* $60521{\cdot}5[1 + {\cdot}001017 \times 36]\log \frac{30}{r}$, corresponding to the barometer reading r. Thus if the two values corresponding to the readings r_u and r_l be deducted from one another we get the value h of the difference in elevation as in equation (2).

Now it must be noted that this table has been compiled for a mean tempera-

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ture 50° F., consequently in considering the corrections due to temperature, it will be necessary to make the correction negative when the mean temperature is below 50° F., and positive when the mean temperature is above 50° F.

For this purpose Table XXI. is compiled from Tables I. and IV. in Appendix X. of the 1881 report of the United States Coast and Geodetic Survey.

TABLE XX.

Containing E (elevation) = $60521{\cdot}5(1 + {\cdot}001017 \times 36° \text{ F.}) \log \frac{30}{r}$

The average temperature $\frac{t_u + t_l}{2}$ is assumed to be 50° F.

r inches.	E feet.	Diff. for ·01. feet.	r inches.	E feet.	Diff. for ·01. feet.	r inches.	E feet.	Diff. for ·01. feet.
19·0	12,445	— 14·5	23·1	7,121	— 11·7	27·2	2,670	— 10·0
19·1	12,302	— 14·2	23·2	7,004	11·7	27·3	2,570	10·0
19·2	12,160	— 14·2	23·3	6,887	11·7	27·4	2,470	9·9
19·3	12,018	— 14·1	23·4	6,770	11·6	27·5	2,371	9·9
19·4	11,877	— 14·0	23·5	6,654	11·6	27·6	2,272	9·9
19·5	11,737	— 13·9	23·6	6,538	11·5	27·7	2,173	9·8
19·6	11,598	13·9	23·7	6,423	11·5	27·8	2,075	9·8
19·7	11,459	13·8	23·8	6,308	11·4	27·9	1,977	9·7
19·8	11,321	13·7	23·9	6,194	11·4	28·0	1,880	9·7
19·9	11,184	13·7	24·0	6,080	11·3	28·1	1,783	9·7
20·0	11,047	— 13·6	24·1	5,967	11·3	28·2	1,686	9·7
20·1	10,911	13·5	24·2	5,854	11·3	28·3	1,589	9·6
20·2	10,776	13·4	24·3	5,741	11·2	28·4	1,493	9·6
20·3	10,642	13·4	24·4	5,629	11·1	28·5	1,397	9·5
20·4	10,508	13·3	24·5	5,518	11·1	28·6	1,302	9·5
20·5	10,375	13·3	24·6	5,407	11·1	28·7	1,207	9·5
20·6	10,242	13·2	24·7	5,296	11·0	28·8	1,112	9·4
20·7	10,110	13·1	24·8	5,186	10·9	28·9	1,018	9·4
20·8	9,979	13·1	24·9	5,077	10·9	29·0	924	9·4
20·9	9,848	13·0	25·0	4,968	10·9	29·1	830	9·4
21·0	9,718	12·9	25·1	4,859	10·8	29·2	736	9·3
21·1	9,589	12·9	25·2	4,751	10·8	29·3	643	9·3
21·2	9,460	12·8	25·3	4,643	10·8	29·4	550	9·2
21·3	9,332	12·8	25·4	4,535	10·7	29·5	458	9·2
21·4	9,204	12·7	25·5	4,428	10·7	29·6	366	9·2
21·5	9,077	12·6	25·6	4,321	10·6	29·7	274	9·2
21·6	8,951	12·6	25·7	4,215	10·6	29·8	182	9·1
21·7	8,825	12·5	25·8	4,109	10·5	29·9	91	— 9·1
21·8	8,700	12·5	25·9	4,004	10·5	30·0	00	9·1
21·9	8,575	12·4	26·0	3,899	10·5	30·1	— 91	9·0
22·0	8,451	12·4	26·1	3,794	10·4	30·2	— 181	9·0
22·1	8,327	12·3	26·2	3,690	10·4	30·3	— 271	9·0
22·2	8,204	12·2	26·3	3,586	10·3	30·4	— 361	9·0
22·3	8,082	12·2	26·4	3,483	10·3	30·5	— 451	8·9
22·4	7,960	12·2	26·5	3,380	10·3	30·6	— 540	8·9
22·5	7,838	12·1	26·6	3,277	10·2	30·7	— 629	8·8
22·6	7,717	12·0	26·7	3,175	10·2	30·8	— 717	8·8
22·7	7,597	12·0	26·8	3,073	10·1	30·9	— 805	8·8
22·8	7,477	11·9	26·9	2,972	10·1	31·0	— 893	
22·9	7 358	— 11·9	27·0	2,871	10·1			
23·0	7,239	— 11·8	27·1	2,770	— 10·0			

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TABLE XXI.

COEFFICIENTS FOR CORRECTING ELEVATION FROM TABLE XX. FOR TEMPERATURE AND HUMIDITY.

$t_w + t_l$	Coefficients.	$t_w + t_l$	Coefficients.	$t_w + t_l$	Coefficients.
0°	− ·103	65°	− ·033	125°	+ ·032
5°	− ·097	70°	− ·027	130°	+ ·037
10°	− ·092	75°	− ·022	135°	+ ·042
15°	− ·086	80°	− ·017	140°	+ ·047
20°	− ·081	85°	− ·011	145°	+ ·052
25°	− ·075	90°	− ·006	150°	+ ·058
30°	− ·070	95°	− ·000	155°	+ ·063
35°	− ·065	100°	+ ·005	160°	+ ·068
40°	− ·059	105°	+ ·010	165°	+ ·073
45°	− ·054	110°	+ ·016	170°	+ ·078
50°	− ·049	115°	+ ·021	175°	+ ·083
55°	− ·043	120°	+ ·026	180°	+ ·088
60°	− ·038				

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APPENDIX II

Transit of Big Loads

The Railway Engineer is frequently requested by the Traffic Superintendent to advise as to the method of loading and hauling such loads of inordinate length and bulk, as are offered to them for transit from time to time by traders, accordingly a few observations on some of the essential points to be kept in mind are here set out.

For what might be termed everyday traffic purposes, all articles are required to be loaded in such a manner that they shall not project in any part beyond the "Maximum Loading Gauge," and it is well, therefore, always to have a diagram with dimensions showing precisely what this gauge is, as they vary slightly on many railway systems. On some railways it is 13′ 6″ above the centre of the four-foot, and on others it is less.

There is another diagram of equal importance and closely allied to it called the "Minimum Structure Gauge," which, as its name implies, controls the nearest points either over or alongside the railway to which any bridge or structure can now be built, and this is 14′ 3″ above the centre of the four-foot.

Unfortunately, on many of the older railways there are over-bridges which do not conform to this gauge, and there are also girders of under-bridges which project considerably above the rails; these fouling points must be carefully recorded, and kept in mind when considering the question of haulage of excessive loads.

It is not always possible to find an absolutely correct detailed drawing of all the older bridges and structures under the control of the Department, and as it would entail too much time on the usually limited staff to make surveys and cross-sections in all these cases, it should be considered by what means sufficient information can be obtained in the shortest possible time, to enable a decision to be come to upon some scheme of loading.

Device for Measuring Existing Structures.—A method which the authors have found successful is shown in Fig. 135, where a home-made apparatus is attached to the end of an ordinary brake van, and used for the purpose of measuring structures very quickly when found to be within the limits. The apparatus consists merely of strongly hinged arms—not shown on the figure—secured to a frame, the points of the arms being so arranged as to conform strictly to the outline of the "Minimum Structure Gauge."

The van is then attached to an engine, and run specially over the district on a selected quiet day—usually, Sundays are the most convenient—when careful observation is kept on the pointers during the run. In all cases where the pointers are disturbed a stop is made, and the draughtsman—properly equipped with spirit level, measuring staff, plumb-bob, tape, etc.—will take

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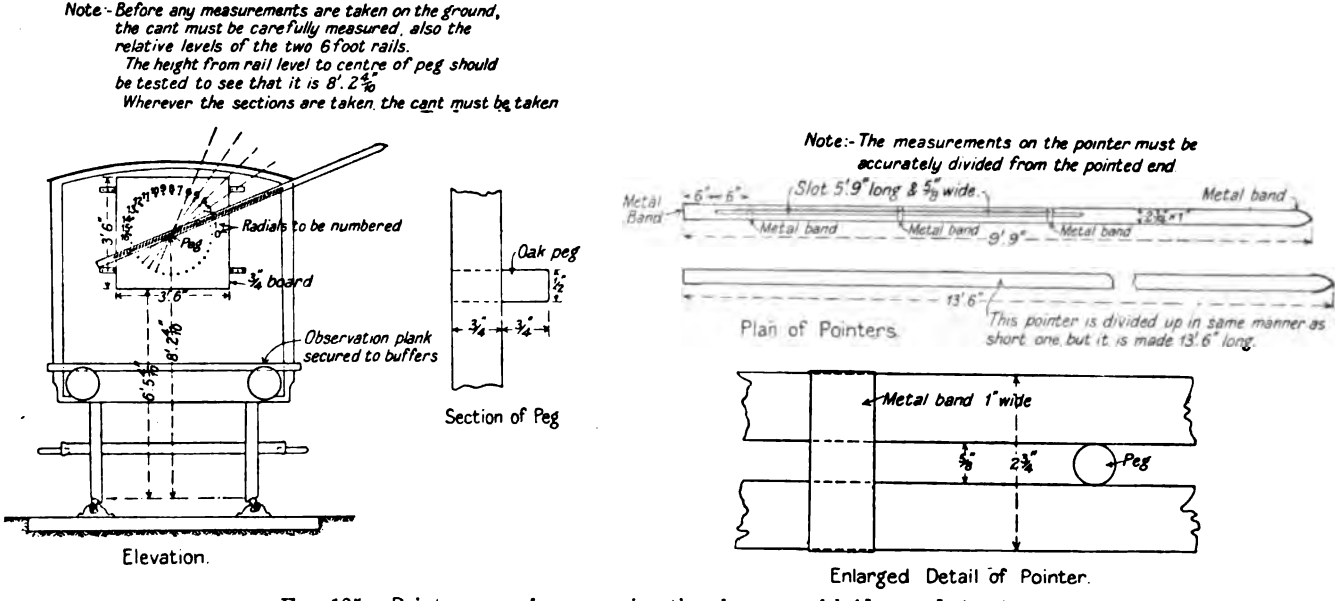
Note:- Before any measurements are taken on the ground, the cant must be carefully measured, also the relative levels of the two 6 foot rails.
The height from rail level to centre of peg should be tested to see that it is 8'. 2 5/16"
Wherever the sections are taken, the cant must be taken
Radials to be numbered
Peg
3/4" board
3'6"
3'6"
8'2 5/16"
6'5 5/16"
Observation plank secured to buffers
Elevation.
Oak peg
1 1/2"
3/4"
3/4"
Section of Peg
Note:- The measurements on the pointer must be accurately divided from the pointed end
Metal Band
6" 6"
Slot 5'9" long & 5/8" wide.
Metal band
Metal band
Metal band
Metal band
9'9"
2 3/4"
1"
13'6"
Plan of Pointers.
This pointer is divided up in same manner as short one, but it is made 13'6" long.
Metal band 1" wide
5/8"
2 3/4"
Peg
Enlarged Detail of Pointer.

FIG. 135.—Pointer gauge for measuring the clearance of bridges and structures.

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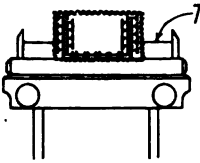
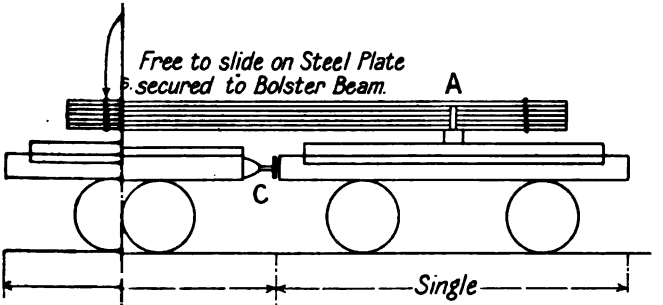


ion showing Bolster at B B.



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particulars of the "tight" places in a comparatively short time, always remembering that the rail level is the *datum* from which the information should be obtained, and that the cant on the road should be taken very accurately at each place where the cross-sectional measurements are required.

A careful record is thereby obtained of all encroachments on the "Minimum Structure Gauge" throughout the district for which the Engineer is responsible, and these records should be plotted in a book, or on drawing paper to a fairly large scale for future reference, as definite advice on such questions is frequently demanded by the Traffic people, and expected to be given in the course of a few hours.

Now all these precautions are quite unnecessary if the load is an ordinary one, such as comes within the "Maximum Load Gauge," but serious questions of transit arise when traders are desirous of forwarding articles considerably outside this gauge, such as large boilers, cables on drums, armour plates, forgings, etc., and it is on these special occasions that the careful records previously referred to are essential, for the reason that an article when loaded must be quite clear of all those tight places, and a scheme of loading must be evolved which will ensure safe transit.

Therefore, with ordinary care, there is not much difficulty in devising a method of framing for supporting a load of excessive bulk, and it remains for the Engineer to state what precautions shall be observed during transit, especially in respect of speed restrictions.

Of course no hard-and-fast rule can be suggested, as every one of these special loads differ in some respects and therefore require individual treatment, but when the question of loading and conveying very long articles comes to be considered, there are certain precautions which must be observed in order to secure safe travelling.

Examples.—The railway companies as a rule have a stock of wagons specially designed for the exclusive purpose of conveying rails, but as the precautions which are about to be enunciated have been incorporated in the equipment of such special vehicles, there is no need to describe them.

Rails being of a flexible nature, it follows that they require supporting at intervals throughout their length, somewhat as shown in Fig. 136, but it is important to remember that there is a great distinction between supporting and fixing.

The load—which in this case consists of a large number of rails suitably stacked and tightly chained at intervals—is prevented from moving laterally by the timber blocks which are spiked to the bolster beams B B, but in order to prevent undue friction the bolster beams are permanently attached to the body of the wagon by means of a central bolt which acts as a swivel. In fact large numbers of this type of wagon are in the service of the Goods Departments in regular use on the principal railways.

Fig. 136 shows very clearly that although the load is bound round by a chain it is supported by a bolster at the points A A and free to slide laterally, and the extent of this lateral sliding may be calculated quite readily when the sharpest curve which the train has to negotiate on its journey is known.

The load being fixed centrally over the four-foot at B B, and being perfectly straight from one end to the other, the offset from the curve will be controlled by the distance between B the fixed point and A the sliding point.

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Knowing the curve and then having calculated the offset, by adding to the result half the width of the load a dimension is obtained, which gives at once the minimum allowance for lateral sliding on the bolster A, and to this calculated amount a little should be added as an extra precaution. It is obvious that if the load were fixed to each of the four bolsters in the manner shown at B the rigid load would simply force some of the wagons off the road when passing round a curve of even moderate radius. It cannot be emphasized too much that long loads of this description must not be secured in any way whatever to the vehicles except to the bolsters only in the two positions indicated. The amount of freedom to slide on the remaining supports can be ascertained by a simple calculation.

Seeing that the load in this instance is secured to the bolsters at B B, it would be an advantage to screw up the coupling at D fairly tightly, otherwise the load may eventually get out of hand by a movement backwards and forwards.

There is no objection to introducing spring buffers and the loose chain coupling at C C, as the extreme ends of the load are free to slide in any direction on the bolster beams.

The bolster beams have attached to them at each end a "D" hook, and in those positions where the load is described as being fixed, it is understood that the chains, etc., are fixed to those "D" hooks only, and not under any circumstances to the body of the wagon, for the reason that a certain amount of "play" is necessary, and this is obtained by the bolster being arranged on a swivel.

Fig. 137 is a very interesting example of the precautions taken to convey safely on the railway a propeller shaft weighing ten tons a distance of 300 miles. In this instance the load does not appear to have been absolutely fixed in any part of its length, although the slack chains and ropes at each end would no doubt "persuade" it to keep within reasonable limits.

From the general principles of loading to be observed which have already been explained, together with the full particulars endorsed on Fig. 137, there appears to be no necessity for any further comment, except perhaps to call attention to the interesting cradle arrangement introduced at A for the purpose of preventing the shaft turning or rolling.

Fig. 138 is an example showing how to load long timbers and similar articles which are capable of sustaining their own weight between the points of support.

Still adopting the principles already laid down, it may be noticed that the loads are again rigidly secured to the swivelled bolsters by means of packings, and chains passing round the loads only which are attached to the steel "D" hooks at the ends of the bolster beams.

No other attachment whatever is made to any other vehicle at any point, the intervening chains shown on the figure being merely for the purpose of binding the logs together, clear of the wagons.

The low-sided trucks at each end and between the bolster wagons—called "Runner Wagons"—are of course only required for the purpose of spacing during transit.

Fig. 139 (p. 293) shows a method which was adopted for carrying an extraordinary load in the form of a drum weighing between sixty and seventy tons. The total weight including load and trucks amounted to something approaching

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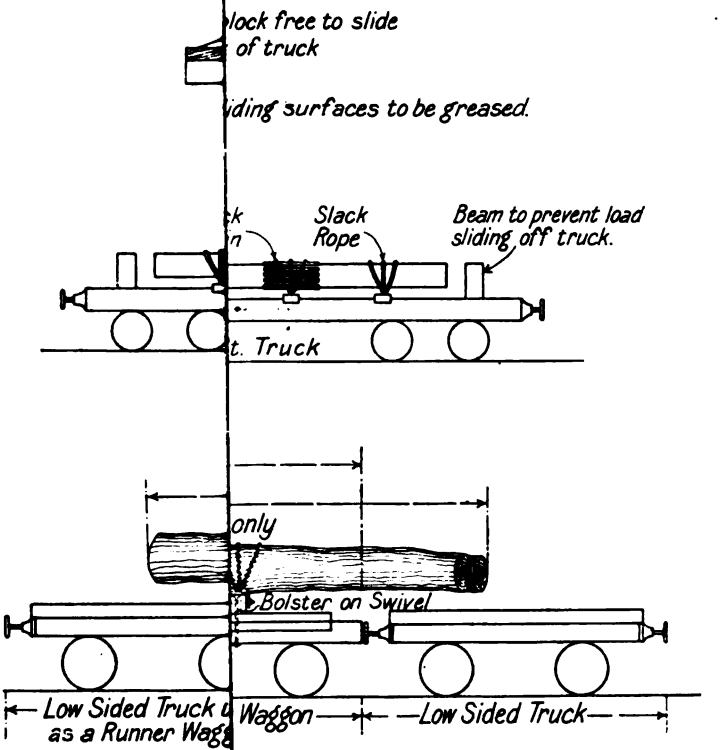

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ninety tons, so that with equal distribution there was a load of nearly fifteen tons on each axle.

The distribution was obtained as far as practicable by means of wedges, which were afterwards spiked and dogged to keep them in position. Arrangements were made for an inspection at important stations during the run of 400 miles, and any slight adjustments which were found necessary were attended to.

Fig. 140 (p. 293) is a further example showing how a steel girder weighing tons may be conveyed.

Very few of the bulky articles—that is those which exceed the "Maximum Load Gauge"—are alike, and it is surprising the number of really interesting problems which fall to the lot of the Railway Engineer, requiring much ingenuity and careful thought in order to solve them satisfactorily.

The mere fact that the Engineering Department frequently have awkward articles of their own to carry about is a sufficient reason to justify the study of this question carefully, apart from the probability of being requested to give expert advice to another department.

Cost of Transit.—The final question of the extra cost to the trader of transit in respect of these exceptional loads is not a matter for the Engineer, provided his obligation is confined to expert advice only, but if the Engineer's Department is involved in charges for loading, examination and so forth, the amounts will be debited to the Traffic people in the usual way, who recover ultimately from the trader.

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TABLE XXII.

This table gives circular curves (of successive inches radius) to any given chains radius for scales of 40, 20, 10, and 8 ft. to an inch.

$$\text{Number of curve (inches in radius)} = \frac{\text{radius in chains} \times 66 \text{ ft.}}{\text{scale in feet}}$$

Chains radius.	40 ft. scale.	20 ft. scale.	10 ft. scale.	8 ft. scale.	Chains radius.	40 ft. scale.	20 ft. scale.	10 ft. scale.	8 ft. scale.
5	8¼	16½	33	41	73	120½	241	481⅝	602
6	10	19⅞	39½	45	74	122½	244½	488⅛	610
7	11½	23	46½	58	75	123⅞	247½	495	619
8	13¼	26½	52⅞	66	76	125½	250⅞	501½	627
9	14⅞	29⅞	59½	74	77	127¼	254	508½	635
10	16½	33	66	83	78	128⅞	257½	514⅞	643
11	18¼	36½	72½	91	79	130½	260⅞	521½	652
12	19⅞	39½	79¼	100	80	132	264	528	660
13	21½	43	85⅞	108	81	133½	267¼	534½	668
14	23	46¼	92½	116	82	135¼	270½	541	676
15	24⅞	49½	99	125	83	136⅞	273⅞	547½	685
16	26½	52⅞	105½	130	84	138½	277¼	544½	693
17	28	56	112¼	140	85	140¼	280½	561	701
18	29⅞	59½	118⅞	145	86	141⅞	283½	567⅞	709
19	31½	62⅞	182	155	87	143½	287	574	718
20	33	66	132	166	88	145¼	290¼	580¼	726
21	34⅞	69¼	138½	174	89	146⅞	293½	587	734
22	36¼	72½	145¼	184	90	148½	297	594	742
23	38	75½	151⅞	190	91	150¼	300½	600½	750
24	39½	79¼	158½	200	92	151½	303½	607	759
25	41¼	82½	165	206	93	153½	306½	613½	767
26	43	85⅞	171½	215	94	155	310	620½	775
27	44½	89	178¼	223	95	156⅞	313½	627	784
28	46¼	92½	184⅞	231	96	158½	316½	633½	792
29	47⅞	95⅞	191½	240	97	160	320	640	800
30	49½	99	198	247	98	161½	323½	646½	808
31	51¼	102½	204½	256	99	163¼	326½	653½	817
32	52⅞	105½	211¼	264	100	165	330	660	825
33	54½	108⅞	217⅞	272	101	166⅞	333¼	666½	833

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34	56	112½	224½	280	102	168½	336½	673½	841
35	57⅝	115½	231	289	103	170	340	680	850
36	59¼	118⅝	237½	297	104	171½	343½	686¼	858
37	61	122	244½	305	105	173¼	346¼	693	866
38	62⅝	125½	250⅝	313	106	175	350	700	874
39	64¼	128½	257½	322	107	176½	353	706¼	883
40	66	132	264	330	108	178¼	356½	712⅝	891
41	67⅝	135¼	270½	338	109	179¾	359½	719½	900
42	69¼	138½	277½	346	110	181½	363	726	907
43	71	142	283¾	355	111	183¼	366¼	732½	916
44	72½	145¼	290½	363	112	184⅝	369½	739¼	924
45	74¼	148½	297	371	113	186½	373	745⅝	933
46	76	151½	303½	379	114	188	376¼	753½	940
47	77½	155	310¼	388	115	189¾	379½	759	949
48	79¼	158½	316⅝	396	116	191½	382⅝	765¼	957
49	80⅝	161¾	323½	404	117	193	386	772½	965
50	82½	165	330	413	118	194⅝	389½	778⅝	973
51	84¼	168½	336½	421	119	196¼	392½	785½	982
52	85⅝	171½	343¼	429	120	198	396	792	990
53	87½	175	349⅝	437	121	199⅝	399½	798	998
54	89	178½	356½	445	122	201½	402⅝	805	1007
55	90⅝	181½	363	454	123	203	406	812	1015
56	92¼	184⅝	369½	462	124	204½	409½	818	1023
57	94	188	376¼	470	125	206¼	412½	825	1031
58	95⅝	191½	382⅝	479	126	208	415⅝	831	1040
59	97½	194⅝	389½	487	127	209⅝	419	838	1048
60	99	198	396	495	128	211½	422½	845	1056
61	100½	201½	402½	503	129	212⅝	425⅝	851	1064
62	102¼	204½	409½	511	130	214½	429	858	1072
63	104	208	415⅝	520	131	216¼	432	864	1081
64	105½	211½	422½	528	132	217⅝	435	871	1089
65	107½	214½	429	536	133	219½	439	878	1097
66	109	217⅝	435½	544	134	221	443	884	1105
67	110½	221	442½	553	135	222½	445	891	1114
68	112¼	224½	448⅝	561	136	224½	449	897	1123
69	113⅝	227⅝	455½	569	137	225⅝	453	904	1130
70	115½	231	462	577	138	227⅝	455	911	1138
71	117½	234½	468½	586	139	229½	459	917	1147
72	118⅝	237½	475½	594					

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TABLE

QUANTITIES IN CUTTINGS AND EMBANK-

Cubic yards per 100 feet slopes: (*a*) ½ to 1;

(*a*) Side slopes ½ to 1.						(*b*) Side slopes ½ to 1.				
Base 16.	Base 18.	Base 20.	Base 28.	Base 30.	Depth d.	Base 16.	Base 18.	Base 20.	Base 28.	Base 30.
60	68	75	105	112	1	61	69	76	106	113
122	137	152	211	226	2	126	141	156	215	230
186	208	230	319	342	3	194	217	239	328	350
252	281	311	430	459	4	267	296	326	444	474
319	356	394	542	579	5	343	380	417	565	602
389	433	478	656	700	6	422	467	511	689	733
460	512	564	771	823	7	506	557	609	817	869
533	593	652	889	948	8	593	652	711	948	1,101
608	675	742	1,008	1,075	9	683	750	817	1,083	1,150
685	759	833	1,130	1,204	10	778	852	926	1,222	1,297
764	845	927	1,253	1,334	11	876	957	1,039	1,305	1,447
844	933	1,022	1,378	1,467	12	978	1,067	1,156	1,511	1,600
926	1,023	1,119	1,505	1,601	13	1,083	1,180	1,276	1,661	1,757
1,010	1,115	1,219	1,633	1,737	14	1,193	1,296	1,400	1,815	1,919
1,096	1,208	1,320	1,764	1,875	15	1,306	1,417	1,528	1,972	2,083
1,184	1,304	1,422	1,896	2,015	16	1,422	1,541	1,659	2,133	2,252
1,274	1,401	1,527	2,031	2,156	17	1,543	1,669	1,794	2,298	2,424
1,366	1,500	1,633	2,167	2,300	18	1,667	1,800	1,933	2,467	2,600
1,460	1,601	1,742	2,305	2,445	19	1,794	1,935	2,076	2,639	2,780
1,555	1,704	1,852	2,444	2,593	20	1,926	2,074	2,222	2,815	2,963
1,653	1,808	1,964	2,586	2,742	21	2,061	2,217	2,372	2,994	3,150
1,752	1,915	2,078	2,730	2,893	22	2,200	2,363	2,526	3,178	3,341
1,853	2,023	2,194	2,875	3,045	23	2,343	2,513	2,683	3,365	3,535
1,955	2,133	2,311	3,022	3,200	24	2,489	2,667	2,844	3,556	3,733
2.060	2,245	2,431	3,171	3,356	25	2,639	2,824	3,009	3,750	3,935
2,166	2,359	2,552	3,322	3,515	26	2,793	2,985	3,178	3,948	4,141
2,274	2,475	2,675	3,475	3,675	27	2,950	3,150	3,350	4,151	4,350
2,384	2,593	2,800	3,630	3,837	28	3,111	3,319	3,526	4,356	4,563
2,496	2,712	2,927	3,786	4,001	29	3,276	3,491	3,706	4,565	4,780
2,610	2,833	3,055	3,944	4,167	30	3,444	3,667	3,889	4,778	5,000
2,726	2,956	3,186	4,105	4,334	31	3,617	3,846	4,076	4,994	5,224
2,844	3,081	3,319	4,267	4,504	32	3,793	4,030	4,267	5,215	5,452
2,964	3,208	3,453	4,431	4,675	33	3,972	4,217	4,461	5,439	5,683
3,085	3,337	3,589	4,596	4,848	34	4,156	4,407	4,659	5,667	5,919
3,208	3,468	3,727	4,764	5,023	35	4,343	4,602	4,861	5,898	6,157
3,333	3,600	3,867	4,933	5,200	36	4,533	4,800	5,067	6,133	6,400
3,460	3,734	4,008	5,105	5,378	37	4,728	5,002	5,276	6,372	6,646
3,589	3,870	4,152	5,278	5,559	38	4,926	5,207	5,489	6,615	6,896
3,719	4,008	4,297	5,453	5,742	39	5,128	5,417	5,706	6,861	7,150
3,852	4,148	4,445	5,630	5,926	40	5,333	5,630	5,926	7,111	7,408
3,986	4,290	4,594	5,808	6,112	41	5,543	5,846	6,150	7,365	7,669
4,122	4,433	4,744	5,989	6,300	42	5,756	6,067	6,378	7,622	7,933
4,260	4,579	4,897	6,171	6,490	43	5,972	6,291	6,609	7,883	8,202
4,400	4,726	5,052	6,356	6,682	44	6,193	6,519	6,844	8,148	8,474
4,541	4,875	5,208	6,542	6,875	45	6,417	6,750	7,083	8,417	8,750
4,684	5,026	5,367	6,730	7,070	46	6,644	6,985	7,326	8,689	9,030
4,830	5,179	5,527	6,919	7,267	47	6,876	7,224	7,572	8,965	9,313
4,978	5,333	5,689	7,111	7,467	48	7,111	7,467	7,822	9,244	9,600
5,127	5,490	5,853	7,305	7,668	49	7,350	7,713	8,076	9,528	9,891
5,278	5,648	6,019	7,500	7,870	50	7,593	7,963	8,333	9,815	10,185

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XXIII.

MENTS (LEVEL SURFACE SECTION).

(*b*) ½ to 1; (*c*) 1 to 1; (*d*) 1½ to 1; (*e*) 2 to 1.

(*c*) Side slopes 1 to 1.						(*d*) Sides slopes 1½ to 1.				
Base 16.	Base 18.	Base 20.	Base 28.	Base 30.	Depth *d*.	Base 16.	Base 18.	Base 20.	Base 28.	Base 30.
63	70	78	107	115	1	65	72	80	109	117
133	148	163	222	237	2	141	156	170	230	244
211	233	256	344	367	3	228	250	272	361	383
296	326	356	474	504	4	326	356	385	568	533
389	426	463	611	648	5	435	472	509	657	694
489	533	578	756	800	6	556	600	644	822	867
596	648	700	907	959	7	687	739	791	998	1,050
711	770	830	1,067	1,126	8	830	889	948	1,185	1,244
833	900	967	1,233	1,300	9	983	1,050	1,116	1,383	1,450
963	1,037	1,111	1,407	1,481	10	1,148	1,222	1,296	1,593	1,667
1,100	1,181	1,263	1,589	1,670	11	1,324	1,406	1,487	1,813	1,894
1,244	1,333	1,422	1,778	1,867	12	1,511	1,600	1,689	2,044	2,133
1,396	1,493	1,589	1,974	2,070	13	1,709	1,806	1,902	2,287	2,383
1,556	1,659	1,763	2,178	2,281	14	1,919	2,022	2,126	2,541	2,644
1,722	1,833	1,944	2,389	2,500	15	2,139	2,256	2,361	2,806	2,917
1,896	2,015	2,133	2,607	2,726	16	2,370	2,489	2,607	3,081	3,200
2,078	2,204	2,330	2,833	2,959	17	2,613	2,739	2,865	3,369	3,494
2,267	2,460	2,533	3,067	3,200	18	2,867	3,000	3,133	3,667	3,800
2,463	2,604	2,744	3,307	3,448	19	3,131	3,270	3,413	3,976	4,117
2,667	2,815	2,963	3,556	3,704	20	3,407	3,556	3,704	4,296	4,444
2,878	3,033	3,189	3,811	3,967	21	3,694	3,850	4,005	4,628	4,783
3,096	3,259	3,422	4,074	4,237	22	3,993	4,156	4,318	4,970	5,133
3,322	3,493	3,663	4,344	4,515	23	4,302	4,472	4,642	5,324	5,494
3,556	3,733	3,911	4,622	4,800	24	4,622	4,800	4,978	5,689	5,867
3,796	3,981	4,167	4,907	5,093	25	4,954	5,139	5,324	6,065	6,250
4,044	4,237	4,430	5,200	5,393	26	5,296	5,489	5,681	6,452	6,644
4,300	4,500	4,700	5,500	5,700	27	5,650	5,850	6,050	6,850	7,050
4,563	4,770	4,978	5,807	6,015	28	6,015	6,222	6,430	7,259	7,467
4,833	5,048	5,263	6,122	6,337	29	6,391	6,606	6,820	7,680	7,894
5,111	5,330	5,556	6,444	6,667	30	6,778	7,000	7,222	8,111	8,333
5,396	5,626	5,856	6,774	7,004	31	7,176	7,406	7,635	8,554	8,783
5,689	5,926	6,163	7,111	7,348	32	7,585	7,822	8,059	9,007	9,244
5,989	6,233	6,478	7,456	7,700	33	8,066	8,256	8,494	9,472	9,717
6,296	6,548	6,800	7,807	8,059	34	8,437	8,689	8,941	9,948	10,200
6,611	6,870	7,130	8,167	8,426	35	8,880	9,139	9,398	10,435	10,694
6,933	7,200	7,467	8,533	8,800	36	9,333	9,600	9,867	10,933	11,200
7,263	7,537	7,811	8,907	9,181	37	9,798	10,072	10,346	11,443	11,717
7,600	7,881	8,163	9,289	9,570	38	10,274	10,556	10,837	11,963	12,244
7,944	8,233	8,522	9,678	9,967	39	10,761	11,050	11,339	12,494	12,783
8,296	8,593	8,889	10,074	10,370	40	11,259	11,556	11,852	13,037	13,333
8,656	8,959	9,263	10,478	10,781	41	11,769	12,072	12,376	13,591	13,894
9,022	9,333	9,644	10,889	11,200	42	12,289	12,600	12,911	14,156	14,467
9,396	9,715	10,033	11,307	11,626	43	12,820	13,139	13,457	14,731	15,050
9,778	10,104	10,430	11,733	12,059	44	13,363	13,689	14,015	15,319	15,644
10,167	10,500	10,833	12,167	12,500	45	13,917	14,250	14,583	15,917	16,250
10,563	10,904	11,244	12,607	12,948	46	14,481	14,822	15,163	16,526	16,867
10,967	11,315	11,663	13,056	13,404	47	15,057	15,406	15,754	17,146	17,494
11,378	11,733	12,089	13,511	13,867	48	15,644	16,000	16,356	17,778	18,133
11,796	12,159	12,522	13,974	14,337	49	16,243	16,606	16,968	18,420	18,783
12,222	12,593	12,963	14,444	14,815	50	16,852	17,222	17,592	19,074	19,444

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TABLE XXIII.—*continued.*

QUANTITIES IN CUTTINGS AND EMBANKMENTS (LEVEL SURFACE SECTION).

Cubic yards per 100 feet.

(*e*) Side slopes 2 to 1.

Depth *d*.	Base 16.	Base 18.	Base 20.	Base 28.	Base 30.
1	67	74	81	111	119
2	148	163	178	237	252
3	244	267	289	378	400
4	356	385	415	533	563
5	481	519	556	704	741
6	622	667	711	889	933
7	778	830	881	1,089	1,141
8	948	1,007	1,067	1,304	1,363
9	1,133	1,200	1,267	1,533	1,600
10	1,333	1,407	1,481	1,778	1,852
11	1,548	1,630	1,711	2,037	2,119
12	1,778	1,867	1,956	2,311	2,400
13	2,022	2,119	2,215	2,600	2,696
14	2,281	2,385	2,489	2,900	3,007
15	2,556	2,667	2,778	3,222	3,333
16	2,844	2,963	3,081	3,556	3,674
17	3,148	3,274	3,400	3,904	4,030
18	3,467	3,600	3,733	4,267	4,400
19	3,800	3,941	4,081	4,644	4,785
20	4,148	4,296	4,444	5,037	5,185
21	4,511	4,667	4,822	5,444	5,600
22	4,889	5,052	5,215	5,867	6,030
23	5,281	5,452	5,622	6,304	6,474
24	5,689	5,867	6,044	6,756	6,933
25	6,111	6,296	6,481	7,222	7,407
26	6,548	6,741	6,933	7,704	7,896
27	7,000	7,200	7,400	8,200	8,400
28	7,467	7,674	7,881	8,711	8,919
29	7,948	8,163	8,378	9,237	9,452
30	8,444	8,667	8,889	9,778	10,000
31	8,956	9,185	9,415	10,333	10,563
32	9,482	9,719	9,956	10,904	11,141
33	10,022	10,267	10,511	11,489	11,733
34	10,578	10,830	11,081	12,089	12,341
35	11,148	11,407	11,667	12,704	12,963
36	11,733	12,000	12,267	13,333	13,600
37	12,333	12,607	12,881	13,978	14,252
38	12,948	13,230	13,511	14,637	14,919
39	13,578	13,867	14,156	15,311	15,600
40	14,222	14,519	14,815	16,000	16,296
41	14,881	15,185	15,489	16,704	17,007
42	15,556	15,867	16,178	17,422	17,733
43	16,244	16,563	16,881	18,156	18,474
44	16,948	17,274	17,600	18,904	19,230
45	17,667	18,000	18,333	19,667	20,000
46	18,400	18,741	19,081	20,444	20,785
47	19,148	19,496	19,844	21,237	21,585
48	19,911	20,267	20,622	22,044	22,400
49	20,689	21,052	21,415	22,867	23,230
50	21,481	21,852	22,222	23,704	24,074

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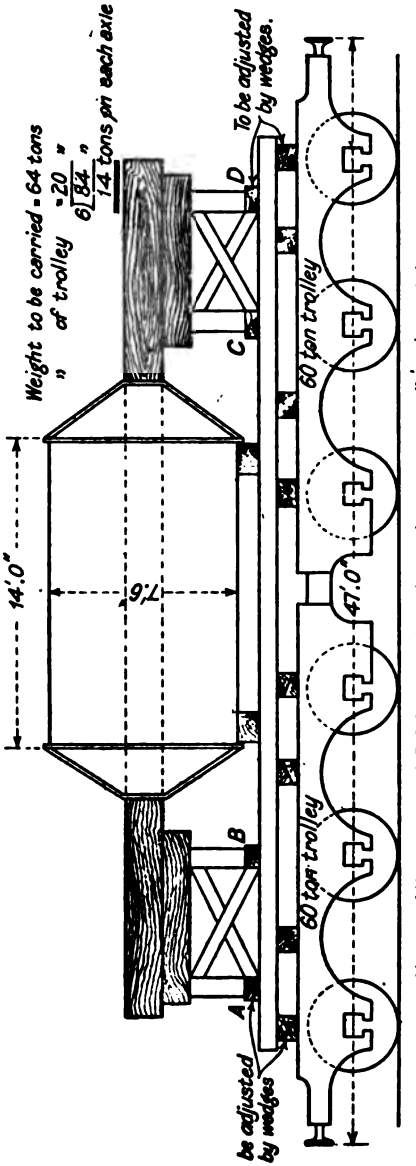

FIG. 139.—Drum carrying wire cable.

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TABLE XXIV.

This table gives offsets from the tangent line at distances equal to the length of the switches, viz. 20ft., 18 ft., 15 ft., 12 ft. and 9 ft. respectively from 5 chains to 480 chains radius. Used in conjunction with article commencing on p. 209, and obtained by the formula $0 = \frac{L^2}{2R}$. *Example.*—Required the offset from the tangent to a 12-chain curve at a distance of 12 ft. from the tangent point.

$$0 = \frac{L^2}{2R} = \frac{12' \times 12' \times 12}{2 \times 12' \times 66} = 1{\cdot}09 \text{ inches.}$$

Radius in chains.	20 ft.	18 ft.	15 ft.	12 ft.	9 ft.	Radius in chains.	20 ft.	18 ft.	15 ft.	12 ft.	9 ft.
	ins.	ins.	ins.	ins.	ins.		ins.	ins.	ins.	ins.	ins.
5	7·2720	5·8908	4·0908	2·6172	1·4724	61	·5964	·4836	·3348	·2148	·1200
6	6·0600	4·9080	3·4080	2·1816	1·2264	62	·5862	·4746	·3300	·2106	·1188
7	5·1948	4·2072	2·9220	1·8696	1·0512	63	·5772	4674	·3246	·2077	·1168
8	4·5444	3·6816	2·5632	1·6356	·9204	64	·5680	·4602	·3204	2044	·1150
9	4·0404	3·2724	2·2728	1·4545	·8184	65	·5599	·4531	·3146	·2014	·1133
10	3·6360	2·9448	2·0460	1·3080	·7368	66	·5509	·4455	·3098	·1983	·1114
11	3·3057	2·6776	1·8594	1·1900	·6694	67	·5424	·4392	·3048	·1944	·1092
12	3·0300	2·4540	1·7040	1·0908	·6132	68	·5346	·4329	·3009	·1923	·1083
13	2·7996	2·2656	1·5732	1·0068	·5664	69	·5268	·4268	·2964	·1896	·1064
14	2·5974	2·1036	1·4610	·9348	·5256	70	·5196	·4207	·2916	·1869	·1044
15	2·4240	1·9632	1·3632	·8712	·4920	71	·5112	·4140	·2880	·1836	·1032
16	2·2722	1·8408	1·2816	·8178	·4602	72	·5050	·4090	·2840	·1818	·1022
17	2·1384	1·7316	1·2036	·7692	·4332	73	·4980	·4032	·2796	·1788	·1008
18	2·0202	1·6362	1·1364	·7272	·4092	74	·4914	·3978	·2766	·1770	·0996
19	1·9140	1·5504	1·0764	·6888	·3876	75	·4848	·3927	·2727	·1745	·0985
20	1·8180	1·4724	1·0224	·6540	·3672	76	·4785	·3876	·2691	·1722	·0969
21	1·7316	1·4024	·9740	·6232	·3504	77	·4722	·3824	·2655	·1699	·0955
22	1·6528	1·3388	·9297	·5950	·3347	78	·4666	·3776	·2622	·1678	·0944
23	1·5804	1·2804	·8893	·5688	·3192	79	·4596	·3720	·2592	·1656	·0924
24	1·5151	1·2275	·8520	·5454	·3066	80	·4536	·3684	·2556	·1622	·0912
25	1·4545	1·1781	·8181	·5234	·2955	81	·4489	·3636	·2525	·1616	·0909

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26	1·3998	1·1328	·7866	·5034	·2832	82	·4434	·3588	·2496	·1596	·0894
27	1·3468	1·0908	·7576	·4848	·2728	83	·4380	·3540	·2460	·1572	·0888
28	1·2987	1·0518	·7305	·4674	·2628	84	·4329	·3506	·2435	·1558	·0876
29	1·2540	1·0152	·7076	·4512	·2544	85	·4277	·3463	·2407	·1538	·0868
30	1·2120	·9816	·6816	·4356	·2460	86	·4224	·3426	·2376	·1518	·0858
31	1·1724	·9492	·6600	·4212	·2376	87	·4180	·3384	·2359	·1504	·0846
32	1·1361	·9204	·6408	·4089	·2301	88	·4131	·3346	·2330	·1486	·0835
33	1·1019	·8925	·6198	·3966	·2231	89	·4080	·3312	·2292	·1464	·0828
34	1·0692	·8658	·6018	·3846	·2166	90	·4044	·3264	·2272	·1452	·0816
35	1·0392	·8400	·5832	·3739	·2088	91	·3999	·3236	·2247	·1439	·0809
36	1·0101	·8181	·5682	·3636	·2046	92	·3951	·3201	·2223	·1422	·0798
37	·9828	·7956	·5532	·3540	·1992	93	·3908	·3164	·2200	·1404	·0792
38	·9570	·7752	·5382	·3444	·1938	94	·3870	·3132	·2172	·1392	·0780
39	·9332	·7552	·5244	·3356	·1888	95	·3828	·3101	·2153	·1377	·0775
40	·9084	·7356	·5088	·3264	·1836	96	·3787	·3068	·2136	·1363	·0767
41	·8868	·7176	·4992	·3192	·1788	97	·3744	·3036	·2112	·1344	·0756
42	·8657	·7011	·4868	·3116	·1752	98	·3710	·3005	·2090	·1335	·0751
43	·8448	·6852	·4752	·3036	·1716	99	·3673	·2974	·2066	·1322	·0744
44	·8264	·6696	·4648	·2975	·1673	100	·3636	·2945	·2045	·1308	·0738
45	·8080	·6545	·4545	·2908	·1636	101	·3600	·2916	·2028	·1296	·0732
46	·7902	·6402	·4446	·2844	·1596	102	·3564	·2886	·2006	·1282	·0722
47	·7740	·6264	·4344	·2784	·1560	103	·3528	·2856	·1980	·1272	·0720
48	·7575	·6135	·4260	·2727	·1533	104	·3499	·2832	·1966	·1258	·0708
49	·7421	·6010	·4180	·2671	·1502	105	·3464	·2800	·1944	·1246	·0700
50	·7272	·5890	·4090	·2616	·1464	106	·3426	·2778	·1926	·1236	·0696
51	·7128	·5772	·4012	·2564	·1444	107	·3396	·2748	·1908	·1224	·0684
52	·6999	·5664	·3933	·2517	·1416	108	·3367	·2727	·1893	·1212	·0682
53	·6852	·5556	·3852	·2472	·1392	109	·3324	·2700	·1872	·1200	·0672
54	·6733	·5453	·3787	·2424	·1362	110	·3305	·2677	·1859	·1190	·0669
55	·6610	·5355	·3719	·2379	·1338	111	·3276	·2652	·1844	·1180	·0664
56	·6493	·5259	·3652	·2337	·1314	112	·3246	·2629	·1826	·1168	·0657
57	·6380	·5168	·3566	·2296	·1292	113	·3204	·2604	·1812	·1152	·0648
58	·6270	·5076	·3538	·2256	·1272	114	·3190	·2584	·1794	·1148	·0646
59	·6156	·4992	·3456	·2220	·1248	115	·3161	·2561	·1779	·1137	·0638
60	·6060	·4908	·3384	·2182	·1230	116	·3135	·2538	·1769	·1128	·0636

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TABLE XXIV.—*continued.*

Radius in chains.	20 ft.	18 ft.	15 ft.	12 ft.	9 ft.	Radius in chains.	20 ft.	18 ft.	15 ft.	12 ft.	9 ft.
	ins.	ins.	ins.	ins.	ins.		ins.	ins.	ins.	ins.	ins.
117	·3111	·2517	·1748	·1119	·0629	182	·1999	·1618	·1123	·0719	·0404
118	·3078	·2496	·1728	·1110	·0624	183	·1988	·1612	·1116	·0716	·0400
119	·3060	·2472	·1716	·1092	·0612	184	·1975	·1600	·1111	·0711	·0399
120	·3028	·2452	·1696	·1088	·0612	185	·1965	·1591	·1106	·0708	·0398
121	·3005	·2433	·1690	·1081	·0608	186	·1954	·1582	·1100	·0702	·0396
122	·2982	·2418	·1674	·1074	·0600	187	·1940	·1574	·1093	·0699	·0393
123	·2956	·2392	·1664	·1064	·0596	188	·1935	·1566	·1086	·0696	·0390
124	·2931	·2373	·1650	·1053	·0594	189	·1924	·1558	·1082	·0692	·0389
125	·2909	·2356	·1636	·1046	·0591	190	·1914	·1550	·1076	·0688	·0387
126	·2886	·2337	·1623	·1038	·0584	191	·1903	·1542	·1072	·0684	·0385
127	·2856	·2316	·1608	·1032	·0576	192	·1893	·1534	·1068	·0681	·0383
128	·2840	·2301	·1602	·1022	·0575	193	·1882	·1526	·1062	·0676	·0380
129	·2816	·2284	·1584	·1012	·0572	194	·1872	·1518	·1056	·0672	·0378
130	·2799	·2265	·1573	·1007	·0566	195	·1866	·1510	·1049	·0671	·0377
131	·2777	·2248	·1561	·0999	·0562	196	·1855	·1502	·1044	·0667	·0375
132	·2756	·2231	·1549	·0991	·0558	197	·1845	·1494	·1038	·0664	·0373
133	·2734	·2215	·1529	·0984	·0553	198	·1836	·1487	·1033	·0661	·0372
134	·2712	·2196	·1524	·0972	·0546	199	·1827	·1479	·1027	·0658	·0370
135	·2693	·2181	·1515	·0969	·0545	200	·1818	·1472	·1022	·0654	·0369
136	·2673	·2164	·1504	·0961	·0541	201	·1808	·1464	·1016	·0648	·0366
137	·2653	·2149	·1493	·0955	·0536	202	·1800	·1458	·1014	·0648	·0366
138	·2634	·2134	·1482	·0948	·0532	203	·1791	·1450	·1011	·0644	·0363
139	·2616	·2118	·1470	·0941	·0527	204	·1782	·1443	·1003	·0641	·0361
140	·2598	·2103	·1458	·0934	·0522	205	·1773	·1435	·0998	·0638	·0360
141	·2580	·2088	·1448	·0928	·0520	206	·1764	·1428	·0990	·0636	·0360
142	·2556	·2070	·1440	·0918	·0516	207	·1756	·1422	·0988	·0632	·0356
143	·2538	·2057	·1430	·0913	·0513	208	·1749	·1416	·0983	·0629	·0354
144	·2525	·2045	·1420	·0909	·0511	209	·1740	·1409	·0978	·0626	·0352
145	·2508	·2030	·1415	·0902	·0508	210	·1732	·1400	·0972	·0623	·0348
146	·2490	·2016	·1398	·0894	·0504	211	·1722	·1394	·0968	·0620	·0348

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147	·2473	·2005	·1391	·0890	·0501	212	·1713	·1389	·0963	·0618	·0348
148	·2457	·1989	·1383	·0885	·0498	213	·1704	·1380	·0960	·0615	·0344
149	·2440	·1976	·1371	·0878	·0495	214	·1698	·1374	·0954	·0612	·0342
150	·2424	·1963	·1362	·0872	·0492	215	·1689	·1370	·0950	·0609	·0345
151	·2408	·1950	·1353	·0866	·0488	216	·1683	·1363	·0946	·0606	·0341
152	·2392	·1938	·1345	·0861	·0484	217	·1675	·1356	·0943	·0601	·0339
153	·2376	·1924	·1337	·0855	·0481	218	·1662	·1350	·0936	·0600	·0336
154	·2361	·1912	·1328	·0849	·0477	219	·1660	·1344	·0932	·0596	·0336
155	·2345	·1898	·1320	·0842	·0475	220	·1652	·1338	·0929	·0595	·0334
156	·2333	·1888	·1311	·0839	·0472	221	·1645	·1332	·0926	·0592	·0333
157	·2315	·1874	·1303	·0833	·0467	222	·1638	·1326	·0922	·0590	·0332
158	·2298	·1860	·1296	·0828	·0462	223	·1630	·1320	·0918	·0587	·0330
159	·2284	·1852	·1284	·0824	·0464	224	·1623	·1314	·0913	·0584	·0328
160	·2268	·1842	·1278	·0816	·0456	225	·1616	·1309	·0909	·0581	·0327
161	·2258	·1828	·1270	·0812	·0456	226	·1602	·1302	·0906	·0576	·0324
162	·2244	·1818	·1262	·0808	·0454	227	·1599	·1297	·0902	·0575	·0323
163	·2230	·1806	·1255	·0803	·0450	228	·1595	·1292	·0897	·0574	·0323
164	·2217	·1794	·1248	·0798	·0447	229	·1587	·1286	·0893	·0571	·0321
165	·2203	·1785	·1239	·0793	·0446	230	·1580	·1280	·0889	·0568	·0319
166	·2190	·1770	·1230	·0788	·0444	231	·1574	·1274	·0885	·0566	·0318
167	·2177	·1761	·1223	·0783	·0441	232	·1567	·1269	·0884	·0564	·0318
168	·2164	·1753	·1217	·0779	·0438	233	·1560	·1263	·0879	·0562	·0316
169	·2151	·1742	·1210	·0774	·0436	234	·1555	·1258	·0874	·0559	·0314
170	·2138	·1731	·1203	·0769	·0434	235	·1548	·1252	·0869	·0557	·0312
171	·2127	·1723	·1189	·0765	·0431	236	·1539	·1248	·0864	·0555	·0312
172	·2112	·1713	·1188	·0759	·0429	237	·1532	·1240	·0864	·0552	·0308
173	·2101	·1702	·1184	·0756	·0426	238	·1530	·1236	·0858	·0546	·0306
174	·2090	·1692	·1179	·0752	·0423	239	·1522	·1231	·0853	·0545	·0306
175	·2078	·1680	·1166	·0747	·0417	240	·1514	·1226	·0848	·0544	·0306
176	·2065	·1673	·1165	·0743	·0417	280	·1299	·1051	·0729	·0467	·0261
177	·2052	·1664	·1162	·0740	·0416	320	·1128	·0921	·0639	·0408	·0228
178	·2040	·1656	·1146	·0732	·0414	360	·1011	·0816	·0591	·0363	·0204
179	·2031	·1644	·1141	·0729	·0411	400	·0909	·0736	·0510	·0327	·0184
180	·2022	·1632	·1136	·0726	·0408	440	·0826	·0669	·0465	·0297	·0167
181	·2011	·1625	·1129	·0723	·0406	480	·0757	·0613	·0424	·0272	·0153

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TABLE XXV.

Offsets for Curves from a Chord 66 Feet in Length.

Offset.	Radius in chains.	Offset.	Radius in chains.	Offset.	Radius in chains.	Offset.	Radius in chains.
1/16	1650·00	7/8	84·51	11/16	17·41	7/8	11·16
1/8	825·00	15/16	83·81	3/4	17·22	9	11·00
3/16	550·00	8	83·00	13/16	17·04	1/8	10·85
1/4	412·50	1/16	82·48	7/8	16·86	1/4	10·71
5/16	317·30	1/8	81·73	15/16	16·68	3/8	10·57
3/8	266·13	3/16	81·13	6	16·53	1/2	10·43
7/16	229·17	1/4	80·55	1/16	16·33	5/8	10·29
1/2	201·22	5/16	29·88	1/8	16·17	3/4	10·16
9/16	179·85	3/8	29·36	3/16	16·00	7/8	10·03
5/8	158·65	7/16	28·84	1/4	15·87	10	9·91
11/16	144·73	1/2	28·35	5/16	15·69	1/8	9·78
3/4	133·07	9/16	27·81	3/8	15·53	1/4	9·66
13/16	123·20	5/8	27·42	7/16	15·37	3/8	9·55
7/8	114·58	11/16	26·87	1/2	15·23	1/2	9·41
15/16	105·77	7/8	26·40	9/16	15·09	5/8	9·32
1	99·89	3/4	25·99	5/8	14·94	3/4	9·22
1/16	98·18	13/16	25·55	11/16	14·80	7/8	9·11
1/8	88·78	15/16	25·13	3/4	14·67	11	9·01
3/16	84·18	4	24·77	13/16	14·54	1/8	8·90
1/4	80·09	1/16	24·37	7/8	14·40	1/4	8·80
5/16	75·68	1/8	24·00	15/16	14·27	3/8	8·71
3/8	72·37	3/16	23·64	7	14·15	1/2	8·61
7/16	68·86	1/4	23·30	1/16	14·02	5/8	8·52
1/2	66·58	5/16	22·96	1/8	13·90	3/4	8·43
9/16	63·46	3/8	22·63	3/16	13·78	7/8	8·34
5/8	61·11	7/16	22·31	1/4	13·66	12	8·25
11/16	58·67	1/2	22·06	5/16	13·54	1/4	8·09
3/4	56·89	9/16	21·71	3/8	13·43	1/2	7·98
13/16	54·62	5/8	21·40	7/16	13·32	3/4	7·77
7/8	52·90	11/16	21·12	1/2	13·20	13	7·62
15/16	51·24	3/4	20·84	9/16	13·10	1/4	7·48
2	49·70	13/16	20·57	5/8	12·99	1/2	7·34
1/16	48·00	7/8	20·31	11/16	12·88	3/4	7·21
1/8	46·61	15/16	20·05	3/4	12·78	14	7·08
3/16	45·33	5	19·80	13/16	12·68	1/4	6·95
1/4	44·00	1/16	19·57	7/8	12·58	1/2	6·84
5/16	42·97	1/8	19·32	15/16	12·47	3/4	6·72
3/8	41·88	3/16	19·09	8	12·38	15	6·61
7/16	40·64	1/4	18·86	1/8	12·19	1/4	6·50
1/2	39·66	5/16	18·64	1/4	12·00	1/2	6·39
9/16	38·78	3/8	18·42	3/8	11·83	3/4	6·29
5/8	37·71	7/16	18·21	1/2	11·65	16	6·19
11/16	36·99	1/2	18·01	5/8	11·49	1/4	6·10
3/4	36·02	9/16	17·81	3/4	11·32	1/2	6·01
13/16	35·20	5/8	17·60				

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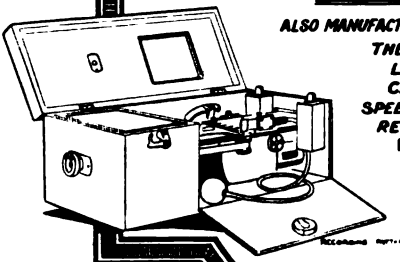

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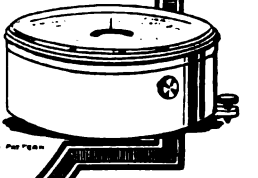

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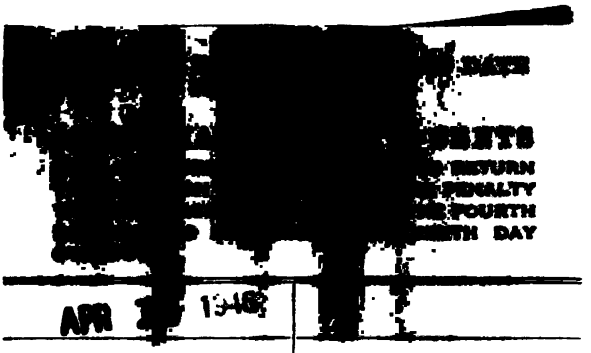

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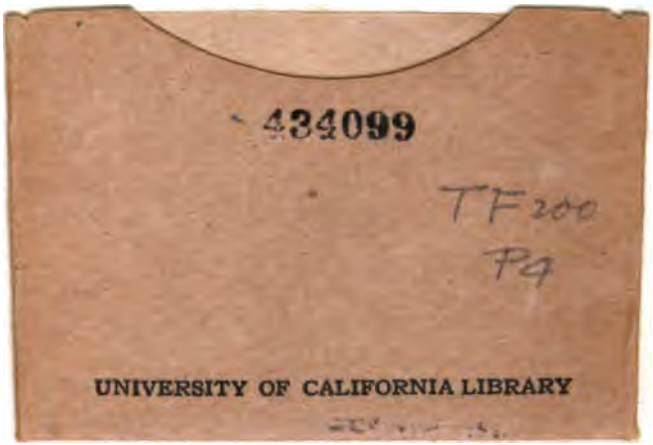

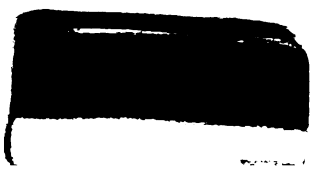

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